

PAPERS, REPORTS, DISCUSSIONS, AND MEMOIRS

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PAPERS AND DISCUSSIONS

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WATER SUPPLY FROM RAINFALL ON VALLEY FLOORS

By A. L. SÖNDEREGGER,* M. Am. Soc. C. E.

SYNOPSIS

This paper presents an analysis of the phenomenon of rainfall penetration on valley floors in semi-arid areas and a discussion of methods for the quantitative determination of the resulting water supply, with special reference to conditions in Southern California.

The following specific examples are discussed:

- 1.—Determination of rainfall penetration from soil moisture tests and from springs in the Murrietta-Temecula Area.
- 2.—Determination of penetration on valley floors by comparison with mountain run-off in San Bernardino Valley.
- 3.—Determination of penetration from rainfall and irrigation rise of water in tests wells in Pauba Valley.

INTRODUCTION

In the semi-arid West, particularly in the valleys of Southern California where seasonal rainfall varies from 6 to 24 in., the water supply resulting from deep penetration of rainfall on the valley floors is of great economic importance.

In the past, engineers have sometimes under-estimated this source of water supply, while in the people's mind the safe yield of ground-water sources has been confused with the capacity of the basins and the supply has been considered unlimited. Recent investigations would indicate that under favorable conditions of rainfall and soil formation the permanent recharge of ground-

NOTE.—The Special Committee on Irrigation Hydraulics has selected the subject of "Evaporation from Soils" as one of ten for study and research. This paper was submitted to the Committee by its author, and the Committee has recommended its publication in *Proceedings* in order to elicit discussion of the subject (see Progress Report of the Committee, *Proceedings*, Am. Soc. C. E., March, 1929, Society Affairs, p. 96). Written discussion on this paper will be closed in September, 1929, *Proceedings*.

* Cons. Engr., Los Angeles, Calif.

water basins from this source may amount to as much as 1 acre-ft. per acre in one season and during a period of years to as much as the supply from the tributary mountain water-shed.

The disposal of rainfall is as follows: (1) Storm run-off; (2) evaporation losses; (3) transpiration losses; and (4) deep penetration. Items (2), (3), and (4) may be grouped under the general term, "absorption", for comparison with Item (1). With closed basins, the resulting water supply represented by Items (1) and (4) may be determined by direct measurement (making allowance for consumptive use and evaporation losses from moist areas); while with open valleys indirect methods may have to be resorted to by assigning values to the different factors. Measurements and other information leading to the determination of any of the factors are, therefore, of general interest. Certain studies made for the alluvial valleys of Southern California, the methods applied, and here presented, may be useful to other investigators.

GENERAL CONDITIONS AFFECTING PENETRATION

Among the general conditions that affect penetration may be listed the character of alluvial deposits, the topography of the ground, the use made of the soil for purposes of agriculture, and the intensity of storms.

Character of Alluvial Deposits.—Seepage water that penetrates intermittently below the limits of evapo-transpiration activity must ultimately cause the wetting—to field capacity—either of the entire formation overlying the water-table, or of well-defined ducts reaching from the surface to the water-table. If there is additional seepage from above, there must be a corresponding addition to the ground-water body.

Alluvial deposits are essentially heterogeneous in formation so that the percolating fluid will naturally find lines of least resistance. Therefore, a uniform saturation over large areas is not likely to occur, but rather the formation of more or less defined ducts. When percolating water strikes an impervious stratum, movement downward is interrupted and a suspended water-table is formed. Gradually, lateral percolation and a circuitous route for the percolating fluid are then established until another pervious deposit again permits a more or less vertical movement. Whenever a suspended water-table has been formed, the drainage from it will occur over the edges of the impervious stratum and is likely to collect into trickling streams, as illustrated in Fig. 1.* These conclusions are supported by the fact that during the process of excavating shafts in alluvial fills it is quite common to strike dry materials alternating with moist materials and occasional streams of water. Inasmuch as alluvial deposits almost invariably show an alternation of pervious and impervious strata, the phenomenon indicated in Fig. 1 is probably that most commonly encountered. Under such conditions, only a part of the alluvial mass between the surface and the ground-water table will be saturated.

The recharge of the ground-water may be hastened or delayed by impervious strata. Large clay bodies may not only cause a regulation of an irregular water supply, but they may also provide storage of a magnitude not appreciated because it is not readily traced.

* The diagrams in this paper were drawn by B. C. Williams, Assoc. M. Am. Soc. C. E.

Absorption is favored by the wedging action of roots, especially decayed roots, which have a tendency to drain a saturated mass along defined canals.

It is concluded, therefore, that both the heterogeneous character of alluvial soils and the action of roots tend to concentrate percolation along lines of least resistance and that a uniform wetting of the soil over large areas to the depth of the root zone is not likely to occur.

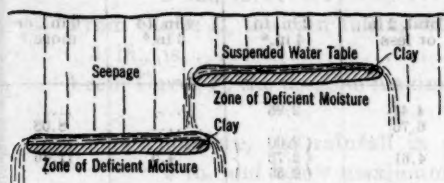


FIG. 1.—EFFECT OF IMPERVIOUS STRATA ON DISPOSAL OF SEEPAGE.



FIG. 2.—CONCENTRATION OF RAIN WATER DUE TO TOPOGRAPHICAL CONDITIONS.

Topographical Conditions.—The effect of a rainstorm is intensified by irregularities of topography, whether they are the gentle undulations which occur on an apparently true plane, or hog-wallows and hummocky undulations, as in Fig. 2. It is apparent that some of the rain falling on Points A and C will flow on the surface to the depression, B. A 3-in. rain may produce at B a depth of water of as much as 6 in. and a seepage greatly in excess of that at Points A and C. This concentration of rain water, therefore, induces a corresponding concentration of seepage and may be responsible, in part, for deep penetration when the seasonal rainfall, uniformly absorbed, is not sufficient to satisfy the moisture deficiency and evapo-transpiration losses.

Intensity of Storms.—The all-important factor appears to be the intensity of storms; that is, rain falling in consecutive days, or within intervals not exceeding two days. Generally speaking, the number and intensity of storms vary with seasonal rainfall. Dry years with rainfall as great as 12 in. are generally devoid of heavy storms and seldom have a storm of more than 3 in. Very wet years generally produce one storm of outstanding intensity and a number of minor storms, although there are exceptions to this rule. Inasmuch as the rate of evaporation is a maximum immediately after a rain, and since even showers foster the growth of grasses, the light rains of a dry winter are, as a rule, immediately wasted, while on the other hand, the lighter storms of a wet winter may saturate the soil to field capacity, and subsequent heavy storms may produce nearly 100% penetration. The superior effects of one heavy storm over a number of light ones becomes magnified when the soil is non-uniform.

Fig. 3 shows the distribution of rainfall into storms of varying amounts based on seasonal and daily records of four stations in the Murrietta-Temecula Area (Riverside County), of Southern California. This is an undulating area, ranging in altitude from 1000 to 1600 ft., with isolated peaks reaching elevations of 3000 ft. It is an interior valley and, comparatively speaking, it is arid (Fig. 4). Seasonal rainfall records are available for a number of stations in the valley area, indicating a long-year mean of 14.5 in., the minima

approaching 6 in. and the maxima, 28 in. The distribution of storms for the station at Wildomar (Elevation 1254), for three characteristic years, is listed in Table 1.

TABLE 1.—RECORD OF STORMS AT WILDOMAR, CALIF.

SEASON.		Seasonal rainfall, in inches.	Total, 2 in., or less.	2 in. to 4 in.*	4 in. to 6 in.*	6 in., or more.*
Year.	Description.					
1923-24	Dry	7.37	4.41	2.96
1916-17	Normal	14.84	6.76	2.00	8.08
1921-22	Wet	27.89	4.61	2.73 2.55	4.5	11.46

* One storm each.

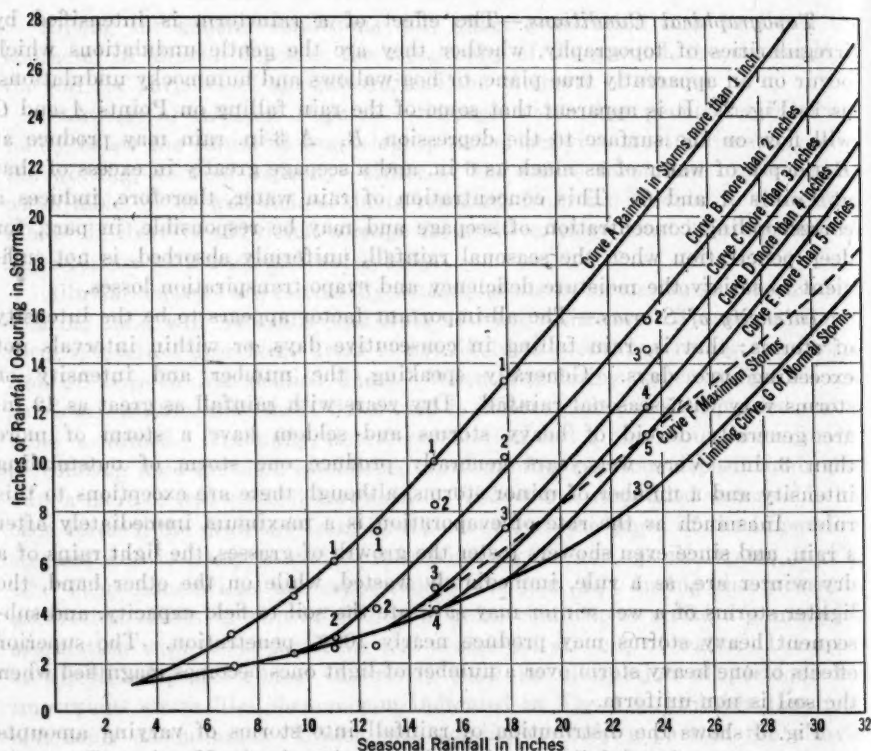


FIG. 3.—DISTRIBUTION OF SEASONAL RAINFALL.

The important feature of this analysis is the illustration of the occurrence each season of one storm of outstanding intensity, necessitating the insertion of a special curve of "maximum storms". Example 1 explains the curves in Fig. 3.

Example 1.—

Given a seasonal rainfall of..... 22.0 in.

From Curve B, the rain falling in storms of more than

2 in., is..... 14.4 "

Therefore, rainfall in storms of less than

2 in., is..... 7.6 in.

From Curve C, the rain falling in storms of more than

3 in., is..... 12.0 in.

From Curve F, the possible maximum rain storm is.... 9.9 "

Therefore, the rainfall in storms of more than

3 in. and below maximum, is..... 2.1 in.

From this it may be concluded that, for a seasonal rainfall of 22 in., there may occur:

One storm of 9.9 in.

Storms from 2 in. to 3 in. (14.4 — 9.9)..... 4.5 "

Storms of less than 2 in..... 7.6 "

Total 22.0 in.

The effect of a series of dry years is to deplete the moisture content to the depth of plant roots, or possibly below that point, evaporation being the minor and transpiration the major factor. It may require a very wet year to recharge a deeply depleted soil. However, if further wet years follow, the amount of deep penetration must assume substantial proportions. This is inferred from a segregation, into storms, of the rainfall at Wildomar (in the arid Murrietta Valley) for three consecutive wet years (see Table 2).

TABLE 2.—ANALYSIS OF RAINFALL AT WILDOMAR, CALIF.

Season.	Seasonal rainfall, in inches.	STORMS OF 2 INCHES, OR MORE.		
		One each, in inches.	Total, in inches.	Percentage of seasonal rainfall.
1913-14	21.33	3.14	15.45	72
		5.50		
		6.81		
		2.73		
1914-15	22.65	2.00	15.20	67
		2.58		
		7.89		
		2.20		
1915-16	21.06	5.15	16.68	80
		9.38		

It is concluded that the major part of deep penetration and recharge from rainfall occurs during a series of consecutive wet years and that the magnitude of the maximum storms may be considered a ruling factor affecting penetration.

Consumptive Use of Cover.—The investigators of the U. S. Department of Agriculture have published the results of great numbers of experiments relative to evaporation and transpiration losses for different types of soils and crops. On the basis of these findings, values for the determination of deep penetration may be assigned to moisture deficiency at the beginning of a rainy season, and to the losses by evaporation, transpiration, and run-off in connection with a study of the individual storms of a season. This method, of necessity, is based on the assumption that rain will remain uniformly distributed over the area after it falls; that the moisture depletion at the end of the season is more or less uniform over the area in question; and that the soil is of uniform texture to the depth to which such depletion occurred. These assumptions, in all probability, will result in excessive values for losses and low values for penetration. On the other hand, if based on daily rainfall records and carried through an entire season, this method will take into account the character of individual storms in a manner no other method will permit, so that the results must be considered as reliable, although too low. Each season, however, must be studied separately.

Harry F. Blaney, Assoc. M. Am. Soc. C. E., has estimated* the total average transpiration for all active growing vegetation in the northern valleys of Southern California for the six winter months (rainy season) as 1 acre-in. per acre per month. For bare lands, vineyards, and deciduous orchards which are clean cultivated, no transpiration losses occur in the winter. Furthermore, the deficiency of soil moisture at the end of the summer is, for various crops:

Irrigated grain and citrus.....	2 to 4 acre-in. per acre
Non-irrigated grain, grass, and weeds..	4 to 6 " " "
Deciduous, grapes, walnuts, peaches..	6 to 8 " " "
Brush	6 to 8 " " "

Evaporation losses are 0.5 in. per acre after each rainstorm, or from 4 in. to 8 in. per season. From these data the water supply that may be required to satisfy the moisture deficiency at the beginning of the rainy season and the evapo-transpiration losses during the same time, is estimated by Mr. Blaney as follows, depending on the wetness of the preceding years and the type of soil:

Bare land.....	6 to 10 in.
Irrigated grain and citrus.....	12 to 16 in.
Non-irrigated grain and weeds.....	12 to 18 in.
Deciduous orchards, grapes.....	10 to 16 in.
Brush	12 to 18 in.

DETERMINATION OF PENETRATION FROM SOIL MOISTURE TESTS ON NORTH MESA, MURRIETTA-TEMECULA AREA

Penetration from rainfall may be ascertained, both as to quantity and depth, by soil moisture tests made before and after a wetting. Test samples are obtained by soil auger or soil tube, and the holes of successive tests as a rule are within a few inches of each other. The test holes are then plugged

* *Bulletin 19*, "Santa Ana River Investigation," State of California (not yet published).

by tamped earth. For fairly uniform soils the results may be considered accurate for the area in the immediate vicinity of the test holes and sufficiently reliable for general conclusions regarding larger areas. In many citrus orchards of Southern California, soil moisture tests are a matter of routine, made to determine the immediate irrigation needs.

TABLE 3.—SEASONAL RAINFALL AND STORM RECORD FOR THE MURRIETTA-TEMECULA AREA, OBSERVED AT WILDOMAR, CALIF.

Season.	Seasonal rainfall, in inches.	Storms of more than 2 in.
1913-14	21.33	3.14*
		5.50
		6.81
		2.73
1914-15	22.65	2.58
		2.00
		7.89
		2.20
1915-16	21.06	5.15
		9.33
		2.75
1916-17	12.26	2.51
1917-18	10.94	3.34
1918-19	7.10
1919-20	11.76	2.64
1920-21	10.10	2.33
		2.45
		2.00
		2.73
1921-22	27.89	2.55
		4.54
		11.46
1922-23	8.37
1923-24	7.37	2.96
1924-25	7.46
1925-26	14.84	8.08
1926-27	20.89	3.08
		10.90

* One storm.

Some irrigation and rainfall penetration tests were made* in February and March, 1927, on the North Mesa of the Murrietta-Temecula Area in Riverside County, Southern California. Fig. 4 is a map of this area. These experiments were made in conjunction with the determination of the water supply available from the rainfall which was indicated by springs in the same general locality. The region is comparatively arid and the soil is an ancient, rather compact alluvium, a combination which would lead the casual observer to the conclusion that no deep penetration could occur. Table 3 gives the seasonal rainfall and storms for a period of years for the station at Wildomar in Murrietta Valley. The altitude of Test Plot No. 1 (see Fig. 4), is 1 400 ft. and the average seasonal rainfall at that place is about 14.5 in. Water was applied by irrigation and by rainfall. Moisture determinations were made from soil-auger samples and were averaged from observations on three holes. The soil is a sandy clay loam, or silt loam, with strata of pure sand at depths of 12 to 15 ft. After a rain, the mesa shows many shallow puddles, from which the water seeps away or evaporates. The land had been dry farmed. The water-table stands at a depth of about 200 ft. The size of test plots was 20 by 20 ft. They were banked on the edges and flooded.

* By Willard G. Babcock, Agricultural Chemist, Riverside, Calif.

The values for Table 4 were obtained from undisturbed cores of soil taken on March 16, 1927. The moisture equivalent test was made to obtain a value for the "specific retention", which is a term used to express the quantity of water which a soil or rock will retain against the pull of gravity if it is drained after having been saturated.*

TABLE 4.—CHARACTERISTICS OF SOIL IN TEST PLOT No. 1.

Factor.	DEPTH BELOW SURFACE:	
	2 ft.	4 ft.
Specific gravity (volume weight).....	1.57	1.81
Wilting coefficient.....	5.8	6.0
Moisture equivalent.....	10.7	11.0
Porosity, in percentage by volume.....	37.2	29.0
Probable yield, in percentage by volume.....	20.3	9.1

The moisture equivalent† is equal to $100 \frac{c}{W}$, in which, c is the weight of the water that remains in the soil after it has been saturated and then subjected to a centrifugal force 1 000 times the force of the gravity, and W is the weight of the soil when dry. It is an arbitrary ratio used in indirect determinations of the hygroscopic and wilting coefficients of soils and may also be found useful for estimating the specific retention.‡

The approximate limits are given by Meinzer§ as follows:

Coarse sand.....	1.5 to 2.5%
Fine sand.....	2.5 to 7.5%
Sandy loam.....	7.5 to 24%
Clay loam.....	24 to 31%

Quoting from a report|| by O. W. Israelsen and F. L. West, Associate Members, Am. Soc. C. E.,

"Correlation between the moisture equivalent and the maximum amounts of water found after irrigation show a gratifying agreement and suggest that the moisture equivalent might be made a basis of judging maximum capillary capacities (essentially specific retention)."

In other words, the moisture equivalent is approximately the field capacity of the soil. The wilting coefficient, as defined by Mr. L. J. Briggs, is equal to the moisture equivalent divided by 1.84.

Table 5 shows the moisture equivalent from determinations made on the test plot at North Mesa (see Fig. 4) during February and March, 1927. Table 6 contains a record of the quantity of water applied on the test plot. Assuming an average specific gravity of 1.69 for the dry soil, 1% of moisture is equivalent

* Water Supply Paper No. 489, U. S. Geological Survey.

† Water Supply Paper No. 494, U. S. Geological Survey, Sheet 12.

‡ Loc. cit., Sheet 11.

§ Water Supply Paper No. 494, U. S. Geological Survey.

|| Bulletin No. 183, Utah Agricultural Experiment Station.

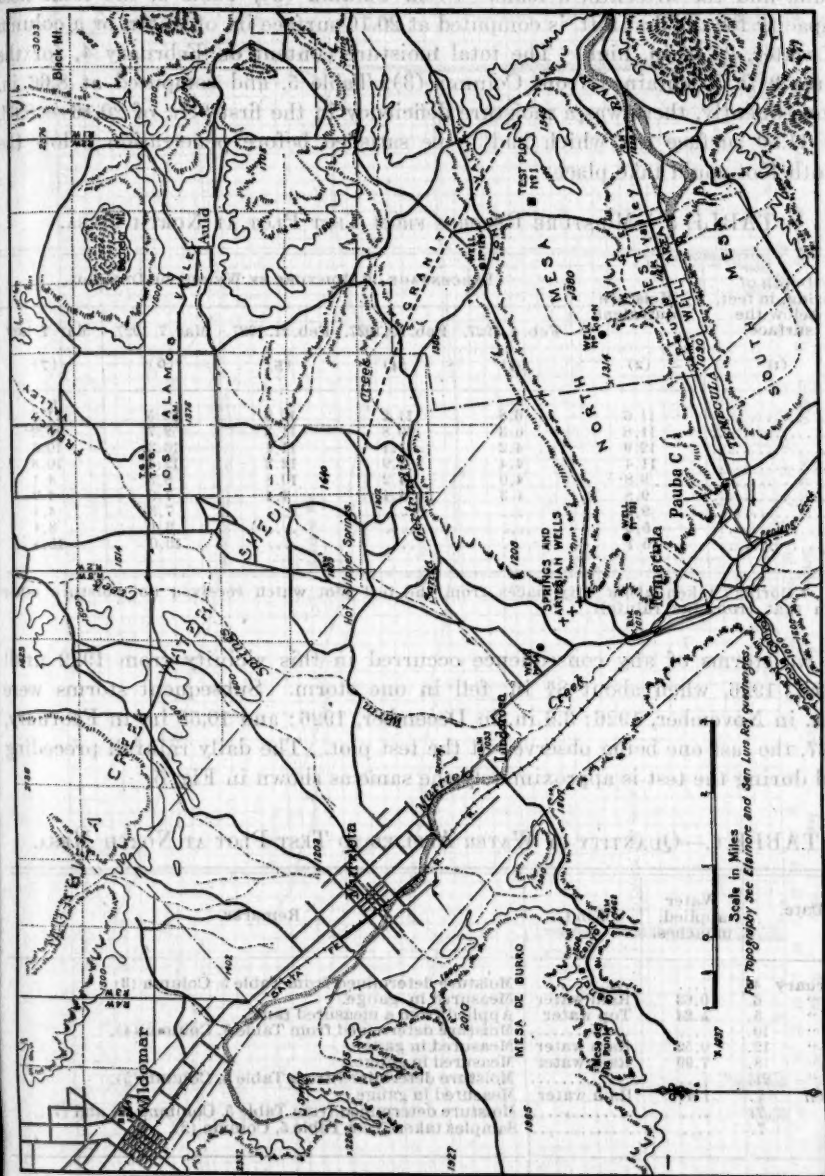


FIG. 4.- MAP OF MURRIETTA-TENEQUILA REGION, CALIFORNIA.

to 0.2028 surface in. of water. The values given in Column (2), Table 5, indicate a heterogeneous soil, the thirteenth foot being sandy, the top foot, a sandy loam, and the fifteenth, a loam. From Column (2), Table 5, the total field capacity for the first 9 ft. is computed at 20.16 surface in. of water, or a column of water, 20.16 in. high. The total moisture content on February 4, for the same 9 ft. is obtained from Column (3), Table 5, and computed at 8.66 in. Consequently, there was a moisture deficiency in the first 9 ft. of 20.16 — 8.66, or 11.50 surface in., which had to be satisfied before penetration below the ninth foot could take place.

TABLE 5.—MOISTURE RECORDS FROM TEST PLOT AT NORTH MESA.

Depth of boring, in feet, below the surface.	Moisture equivalent.	PERCENTAGE OF MOISTURE BY WEIGHT OF DRY SOIL.				
		Feb. 4, 1927.	Feb. 10, 1927.	Feb. 21, 1927.	Mar. 7, 1927.	Mar. 7, 1927.
(1)	(2)	(3)	(4)	(5)	(6)	(7)
1.....	11.6	6.8	11.2	10.9	12.5	11.6
2.....	11.8	6.3	10.8	11.3	9.7	10.2
3.....	12.9	4.2	11.1	11.1	10.3	10.3
5.....	11.4	4.4	4.9	12.3	12.4	10.8
7.....	9.8	4.0	4.2	10.8	8.3	4.1
9.....	9.8	4.3	4.4	4.4	7.0	4.9
11.....	9.2	7.3	4.4
13.....	5.2	3.1	3.4
15.....	15.1	10.0	15.4

* Borings taken about fifty paces from the test plot which received no moisture other than that from the rainfall.

No storms of any consequence occurred in this vicinity from 1922 until April, 1926, when about 8½ in. fell in one storm. Subsequent storms were 1 in. in November, 1926; 3.6 in. in December, 1926; and 10.58 in. in February, 1927, the last one being observed at the test plot. The daily rainfall preceding and during the test is approximately the same as shown in Fig. 5.

TABLE 6.—QUANTITY OF WATER APPLIED TO TEST PLOT AT NORTH MESA.

Date.	Water applied, in inches.	Kind.	Remarks.
February 4.	Moisture determined from Table 5, Column (3).
" 6.	0.63	Rain water	Measured in gauge.
" 5.	4.24	Top water	Applied from a measured tank.
" 10.	Moisture determined from Table 5, Column (4).
" 12.	0.52	Rain water	Measured in gauge.
" 18.	7.99	Rain water	Measured in gauge.
" 21.	Moisture determined from Table 5, Column (5).
March 7.	1.44	Rain water	Measured in gauge.
" 7.	Moisture determined from Table 5, Columns (6) and (7).
" 7.	Samples taken from Table 5, Column (2).
.....	14.82	Total water applied on plot.

From Column (3), Table 5, it is apparent that at the test plot the heavy rain of December, 1926, did not penetrate below the second foot and that as a

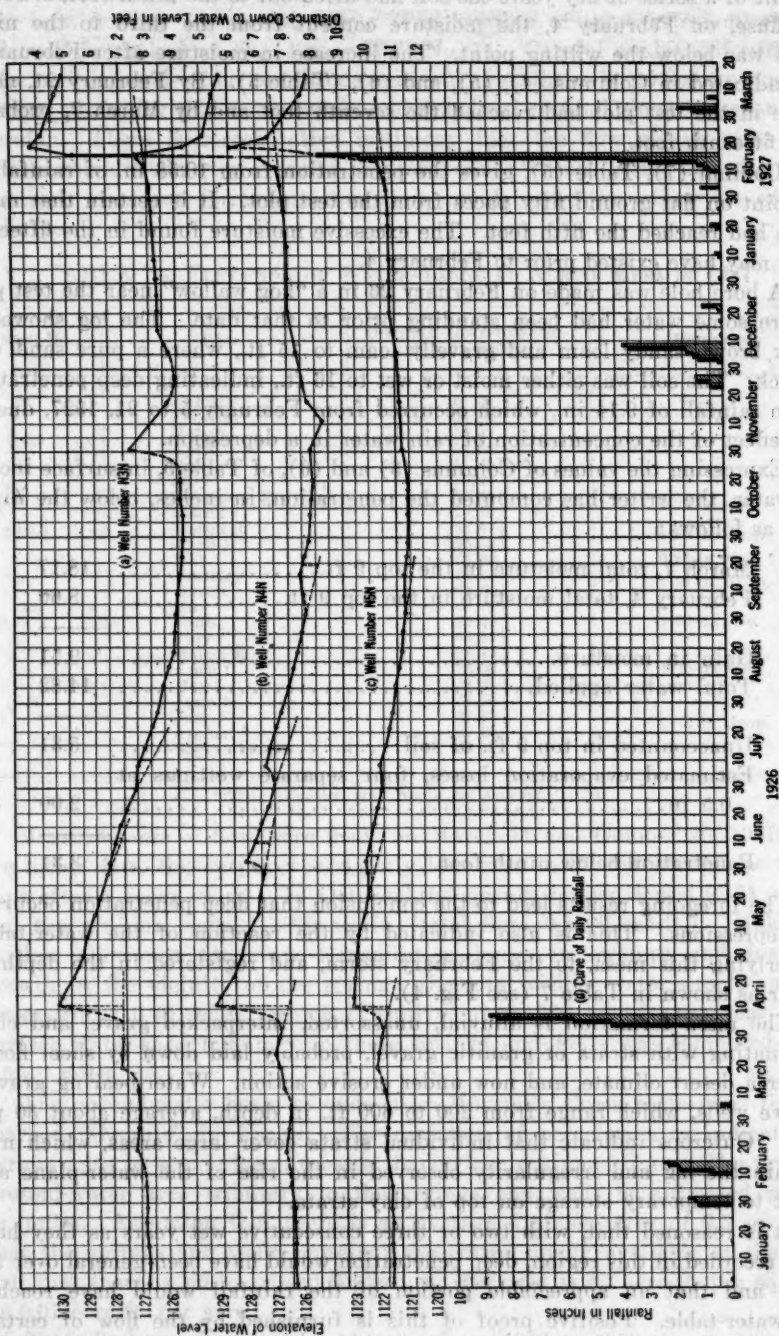


FIG. 5.—FLUCTUATIONS OF WATER LEVELS IN TEST WELLS, PAUBA VALLEY, CALIFORNIA.

result of a series of dry years the soil had dried out to the ninth foot, or lower, because, on February 4, the moisture content from the third to the ninth foot was below the wilting point. The increase in moisture after February 4 is indicated in Columns (4), (5), and (6), (Table 5). By February 21, moisture in the test plot had reached the seventh foot and by March 7, probably the fifteenth foot.

Column (7), Table (5), gives the penetration from 10.58 in. of rainfall at a point on flat ground fifty paces from the test plot. It is certain that moisture had reached the fifth foot. The excessive moisture found in the fifteenth foot may have existed prior to February 4.

A bore hole was made on February 26 in a "hog wallow" near the test plot where some water had been standing prior to that date. The log showed a dark brown sandy loam and gravelly loam to 15 ft., where a pure sand was struck. The soil was either moist or wet to 16 ft., indicating deep penetration from rainfall of 9.14 in., which occurred from February 5 to 21, 1927, due to the effect of the concentration of rain water in a depression.

Expressing the values of Columns (3) and (6), of Table 5, in surface inches of water, the writer has computed the penetration, in inches, below the ninth foot as follows:

March 7, total moisture in the top 9 ft.....	18.17
February 4, total moisture in the top 9 ft.....	8.66
Gain in moisture.....	9.51
Total water applied.....	14.82
Unaccounted in top 9 ft. of soil.....	5.31
Estimated evaporation losses, four separate wettings at 0.5 in.....	2.00
Penetration below ninth foot.....	3.31

The foregoing results lead to the conclusion that deep penetration occurred in depressions. This is also indicated by the reaction of the water-table, underlying this mesa, to the February storm, and registered in the depth to water as shown in Table 7 (see Fig. 4).

The mesa formation is alluvial, unassorted, interpacked gravel and clay, alternating with strata of granitic gravel, probably laid down by sheet floods under a desert climate, and now under erosive action. Water-bearing gravels in five wells, which range from 200 to 600 ft. in depth, average about 30 per cent. Outcrops indicate that individual strata cover large areas, which may explain the lag and irregularity observed in the rise of the water-plane and point to temporary storage on top of clay strata.

It is reasoned that, with two or three consecutive wet years as they have been recorded in this region, deep penetration would have been general over the mesa and that an appreciable portion of the rainfall would have reached the water-table. Positive proof of this is furnished by the flow of certain springs which issue at the foot of the steep southwesterly escarpment of the

North Mesa and by the output of some artesian wells near the springs which have been flowing for several years. The location of these artesian springs and wells is marked on Fig. 4 near Well No. 14. Careful analysis of surface-water and ground-water conditions indicate that there is no extraneous, or distant, source for this water supply and that local rainfall alone is responsible therefor. There is corroborative testimony that the flow of the wells and springs varies little over a period of years, the inference being that the comparatively compact formation acts as a regulator of an otherwise erratic recharge.

TABLE 7.—ELEVATION OF WATER-TABLE AT NORTH MESA IN 1927.

Well No.	Date.	Distance down to water-table, in inches.	Remarks.
129	August*	154.00	
	February	154.16	
	March	154.40	
	April	154.25	
	May	154.46	
	July	154.20	
	February 23	123.48	
	March 9	123.19	0.29-in. rise.
	March 16	123.47	
	April 4	123.55	0.36-in. drop.
LSN	May 1	123.30	0.25-in. rise.
	January	63.39	
	March	64.60	1.31-in. drop.
	May	63.70	
	June	63.40	1.30-in. rise.
	July	63.30	
	March 21	38.49	
	April 1	38.00	1.10-in. rise.
	May 7	37.39	
	June 4	37.52	
14	July 2	37.59	
	August 8	37.86	0.72-in. drop.
	September 24	38.11	

* This reading was made in August, 1926.

Measurements of flow were made on March 11 and October 26, 1927. On the latter date, the output was slightly less than in March which was thirty days after a heavy storm. In October, the springs proper flowed 0.191 sec.-ft., and the artesian wells, 0.1672 sec.-ft. (total flow, 0.3582 sec.-ft.). Making a further allowance for evapo-transpiration losses in swamps and for wild growth in the vicinity of the springs, the total visible water supply may be estimated at 0.40 sec.-ft., which is equivalent to 290 acre-ft. per year. The surface water-shed tributary to these springs and wells was computed to be 5.5 sq. miles.

The ground-water basin extends from Murrietta Creek easterly to the line marked "Granite" in Fig. 4. The direction of percolation is indicated by arrows and, in some places, crosses the surface drainage lines. The probable ground-water area tributary to the springs and wells is estimated at 7.34 sq. miles. Assuming that there is an average drainage area of 6.5 sq. miles, that the springs and wells represent the entire supply available at the bluff, and that there is no by-passed underflow, the yield is 45 acre-ft. per sq. mile, or 0.07 acre-ft. per acre.

Table 3 shows that for the 14 years from 1913 to 1927 the mean rainfall in this vicinity was 14.57 in. and that there were 5 very wet years, with an

average of 22.76 in. (= 1.9 ft.). If the recharge is assumed to have taken place in these 5 years, the mean percentage of deep percolation of a wet year was $\frac{14 \times 0.07}{5 \times 1.9} = 10.3\%$ of the seasonal rainfall of 22.76 in.

DETERMINATION OF DEEP PENETRATION AND STORM RUN-OFF BY COMPARISON WITH MOUNTAIN RUN-OFF

The quantity and general character of the rainfall, as well as the physical characteristics that might be peculiar to an individual mountain water-shed, are expressed in the rainfall run-off relations, and these relations can be applied with modifications to conditions on adjacent alluvial plains. A study is here presented with this end in view and illustrated for a concrete case, namely, the application of the observed rainfall-run-off relations of the San Bernardino Mountain water-shed in Southern California to the determination of deep penetration from rainfall in the San Bernardino Valley.

Reliable run-off measurements of mountain water-sheds are generally available while rainfall records, as a rule, are incomplete. The reverse is the case for valleys. Rainfall increases with altitude, in Southern California, up to about Elevation 6 000 ft. The average rainfall of a water-shed can be determined from available records, either by the construction of isohyets lines or the use of formulas.

Effect of Different Soil Formations.—The modern alluvial valley fill which has its origin in the residual soil of a granitic range is generally more open and, for similar storm and cover conditions, should produce a larger percentage of deep penetration than the residuum. Similarly, the product of erosion of an ancient alluvium will be more porous and subject to more rapid penetration than its mother deposit. An ancient compact alluvium has a larger specific retention than a modern stream deposit and will hold larger volumes of water, subject to evapo-transpiration losses, and, therefore, will produce less available supply. This is demonstrated in Fig. 6 by comparing the run-off curves of the Santa Ana River water-shed with those of the water-sheds of Lytle, Mill, San Antonio, Devil Canyon, and Waterman Canyon Creeks.

The water-shed of Santa Ana Canyon is characterized by flats and valleys covered with an ancient alluvium, while in the water-sheds of the other streams mentioned residual soils and slides are the predominant features, and although the Santa Ana River has a higher mean seasonal rainfall, nevertheless its percentage of run-off is less than in the case of the other water-sheds.

Comparison of Physical Features Affecting Penetration and Run-Off.—On a mountain water-shed with residual soil cover, seepage water may strike bed-rock at a depth of 4 to 12 ft. or more, where it will be diverted down-hill as illustrated in Fig. 7 (a). On this down-hill course it must run the gauntlet of absorption by the roots of trees, brush, and herbs which may have penetrated fissures to depths of 20 to 30 ft. Abstraction is, therefore, extremely active until the percolating water reaches the stream bed. Despite this continued tapping, however, seepage run-off is produced in dry years, although this may be due partly to over-year storage and partly to absorption on slides, which, owing to their unstable and porous formation, readily admit the rain water.

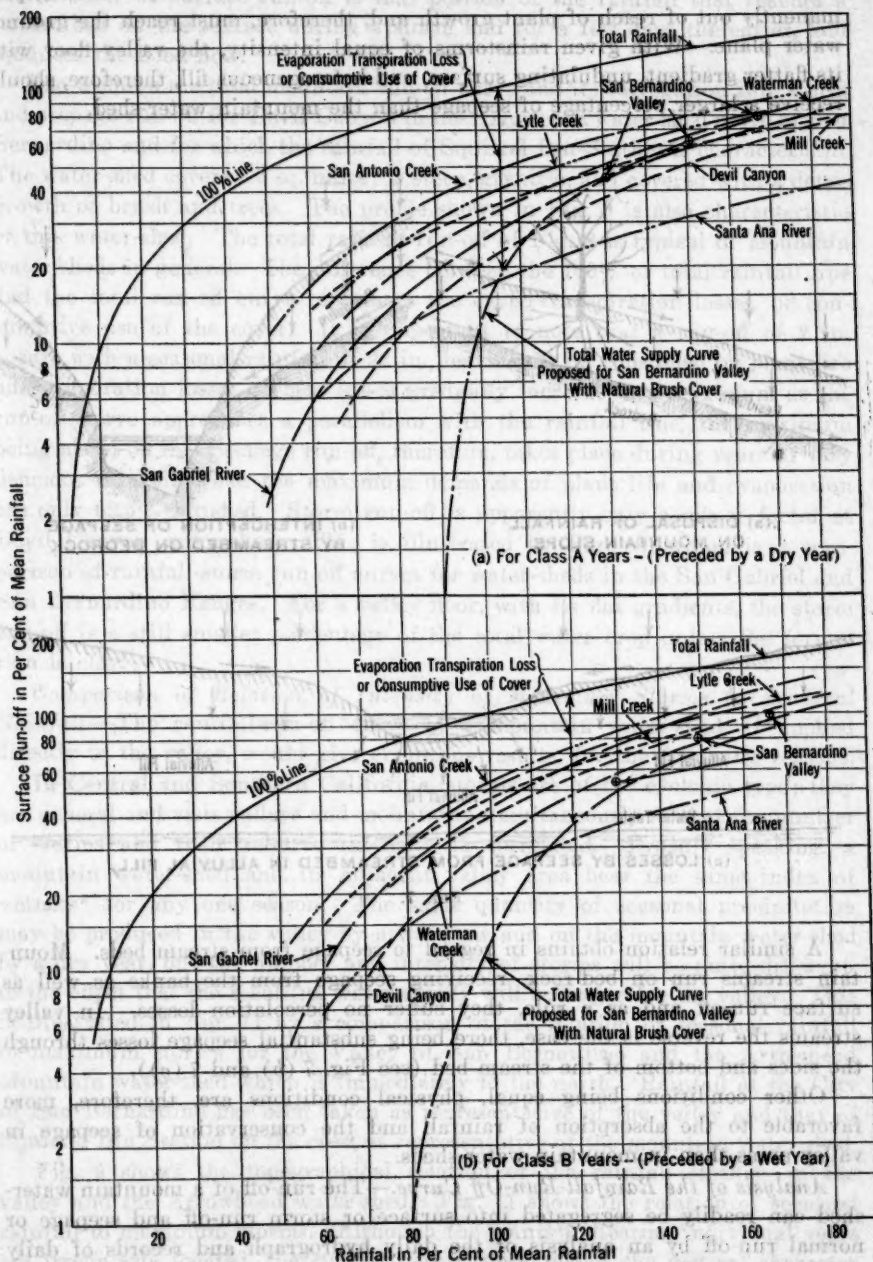


FIG. 6.—RELATION BETWEEN TOTAL SEASONAL RUN-OFF AND SEASONAL RAINFALL.

Different conditions prevail on valley floors. Bed-rock may be at depths of 20 ft. or more, and water that percolates to 10 or 15 ft. is, as a rule, permanently out of reach of plant growth and, therefore, must reach the ground-water plane. With given rainstorms of equal intensity, the valley floor with its flatter gradient, undulating surface, and heterogeneous fill, therefore, should receive a larger percentage of seepage than the mountain water-shed.

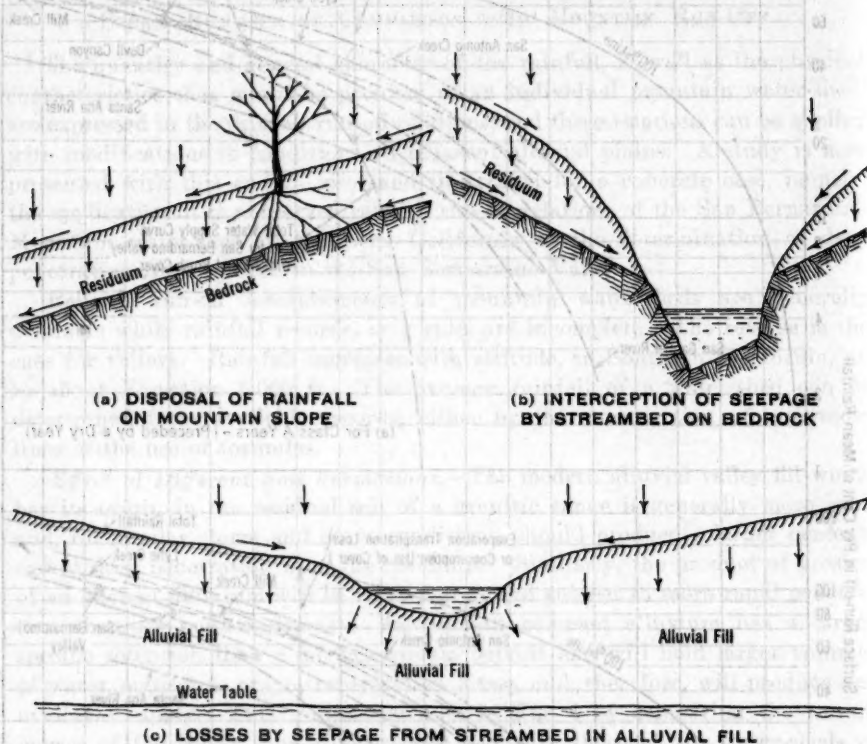


FIG. 7.

A similar relation obtains in regard to seepage from stream beds. Mountain streams run on bed-rock, receiving seepage from the banks as well as surface run-off and as a rule, they suffer no percolation losses. In valley streams the reverse is the case, there being substantial seepage losses through the sides and bottom of the stream bed (see Fig. 7 (b) and 7 (c)).

Other conditions being equal, physical conditions are, therefore, more favorable to the absorption of rainfall and the conservation of seepage in valley areas than in mountain water-sheds.

Analysis of the Rainfall-Run-Off Curve.—The run-off of a mountain watershed can readily be segregated into surface or storm run-off and seepage or normal run-off by an analysis of the daily hydrograph and records of daily discharge. Seepage run-off is defined herein as that portion of the rain which seeps into the ground and, subsequently, is the source of the more or less

perennial flow of a stream; it corresponds to deep penetration in the valley. Storm-water or surface run-off is that portion of the rainfall that reaches a stream bed on the surface during a storm and for a few days thereafter, and produces the flood flow.

Fig. 8 shows the total mountain run-off curve and its segregation into storm and seepage run-off for Devil Canyon in the Arrowhead water-shed north of San Bernardino and for which the rainfall of Squirrel Inn Station is characteristic. The water-shed covers 6.3 sq. miles; is steep, granitic, and covered with a dense growth of brush and trees. The profile shown in Fig. 9 is also characteristic of this water-shed. The total rainfall-run-off of Fig. 8 is typical of mountain water-sheds in general. The difference between the 100% or total rainfall line and the total run-off curve represents the evapo-transpiration losses, or consumptive use of the cover. It is important to note that a run-off of 2 in. occurs with a seasonal rainfall of 22 in., leaving 20 in. consumed by plant life and evaporation losses. These losses gradually increase to a maximum as the run-off curve approaches a parallelism with the rainfall line, the maximum being about 30 in. Seepage run-off, therefore, takes place during years of very deficient rainfall, when the maximum demands of plant life and evaporation are only 62.5% satisfied. Storm run-off is apparently only a minor factor at Devil Canyon water-shed. This is illustrated by Fig. 10, which is a comparison of rainfall-storm run-off curves for water-sheds in the San Gabriel and San Bernardino Ranges. For a valley floor, with its flat gradients, the storm run-off is a still smaller percentage of the total water crop unless the formation is clay.

Comparison of Relation of Intensity of Maximum Storms to Seasonal Rainfall.—The rainfall-run-off curve of a mountain water-shed, if applied directly to the valley, would give erroneous results, for the following reasons.

In Central and Southern California, storms are of the cyclonic type; they are general and visit valleys and mountains simultaneously so that the number of storms and their relative intensity is correlated. Broadly speaking, a mountain water-shed and its adjacent valley area bear the same index of wetness* for any one season. The same quantity of seasonal precipitation may be produced in the valley by a wet year and on the mountain water-shed by a dry year, but the number and intensity of storms will be quite different, to the effect that the resulting water supply will be larger for the valley. This is illustrated in Fig. 11 by a comparison of the relation of seasonal rainfall to maximum storms for the Valley of San Bernardino and the Arrowhead Mountain water-shed which is immediately to the north. Rainfall at the City of San Bernardino has been taken as representative of the valley and that of Squirrel Inn Station on the crest as representative of the mountain water-shed.

Fig. 9 shows the topographical relation of the rainfall stations in the valley and the Arrowhead water-shed. Fig. 11 shows the relation of seasonal rainfall to maximum storms. Although the points indicating individual years are irregularly located, the curves, nevertheless, depict the general character

* The "index of wetness" is the percentage of the long-period mean. A seasonal rainfall of 20 in. would have an index of wetness of 125 for a long-period mean of 16 in.

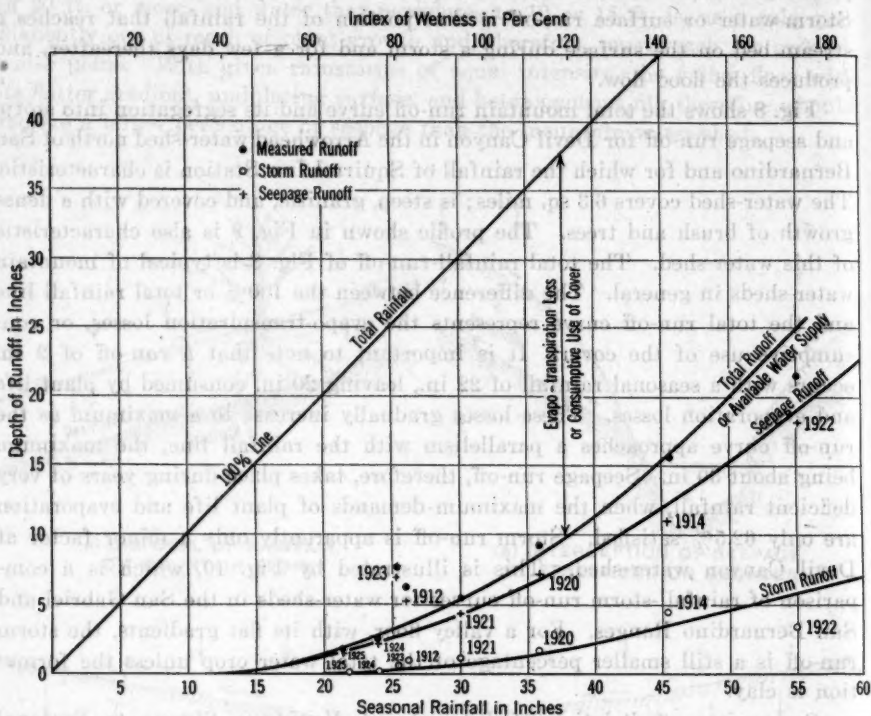


FIG. 8.—RELATION OF SEASONAL RAINFALL TO MEASURED TOTAL RUN-OFF SEEPAGE AND STORM RUN-OFF FOR DEVIL CANYON WATER-SHED.

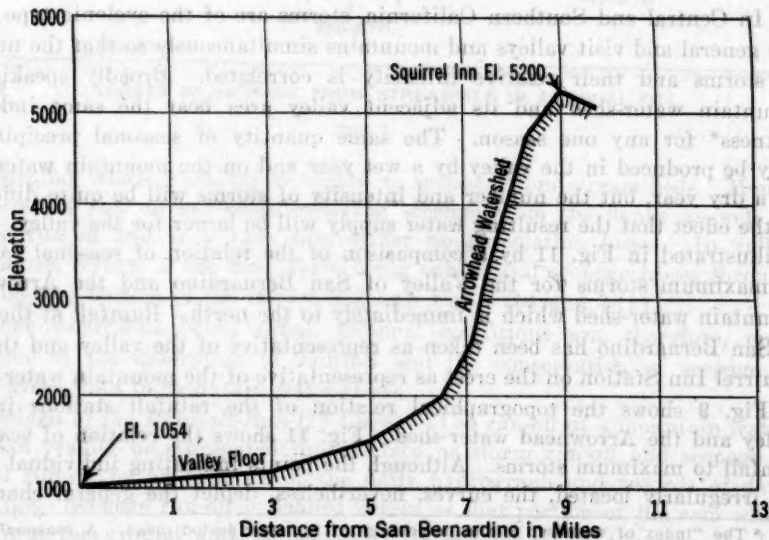


FIG. 9.—PROFILE OF ARROWHEAD WATER-SHED.

of the correlation between mountain and valley rains. For example, in order to produce a 7.5-in. storm a seasonal rainfall of 21 in. is required at San Bernardino and one of 30 in. at Squirrel Inn Station.

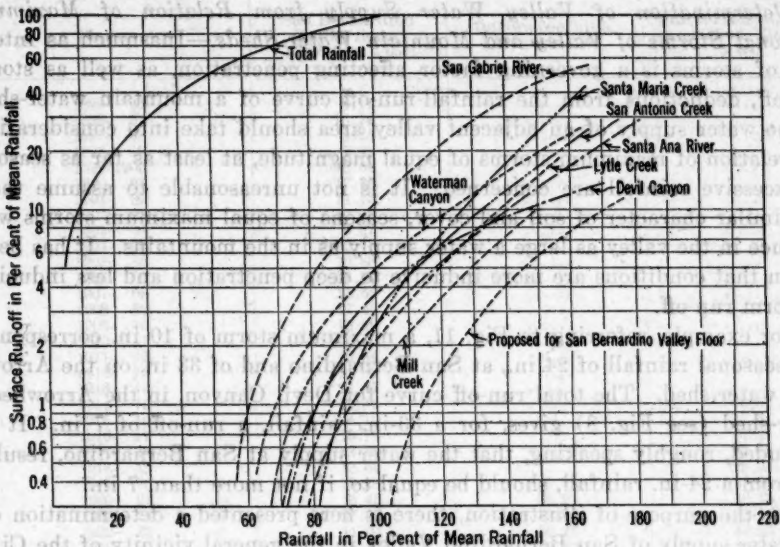


FIG. 10.—COMPARISON OF RAINFALL-STORM RUN-OFF CURVES.

Fig. 12 shows the relation of the indices of wetness to the maximum storms. It is apparent that in the mountains, as well as in the valley, heavy storms do not occur for seasonal rains below normal. In Fig. 11, the dashed line represents the "maximum storm-rainfall curve" for the Arrowhead water-shed near San Bernardino, to which Devil Canyon and Waterman Canyon belong.

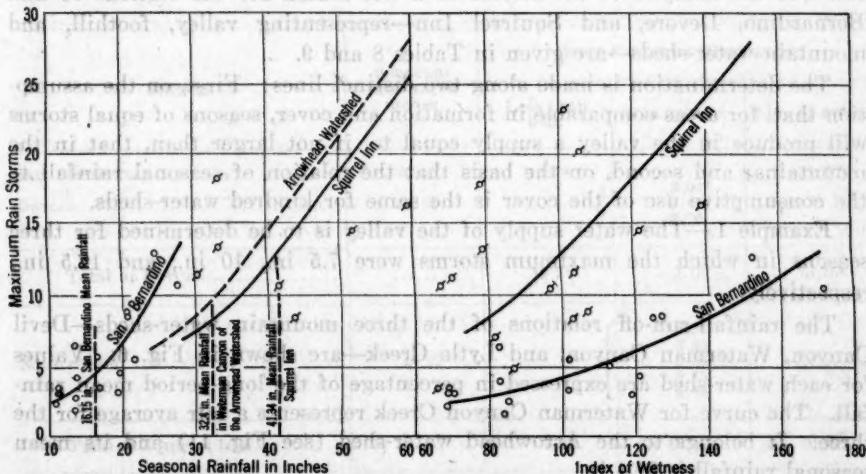


FIG. 11.—RELATION BETWEEN SEASONAL RAINFALL AND MAXIMUM STORMS.

FIG. 12.—RELATION OF INDEX OF WETNESS TO MAXIMUM STORMS.

A study of Figs. 9, 11, and 12, leads to the conclusion that, for equal seasonal rainfall, the valley receives storms of much greater intensity than the mountain water-shed and produces a correspondingly larger water supply.

Determination of Valley Water Supply from Relation of Maximum Seasonal Storms of Valley and Mountain Water-Sheds.—Inasmuch as intensity of storms is a governing factor affecting penetration, as well as storm run-off, deductions from the rainfall-run-off curve of a mountain water-shed to the water supply of an adjacent valley area should take into consideration the relation of maximum storms of equal magnitude, at least as far as seasons of excessive rainfall are concerned. It is not unreasonable to assume that, for similar character of soil and cover, seasons of equal maximum storms will produce in the valley as large a water supply as in the mountains. It has been shown that conditions are more conducive to deep penetration and less conducive to storm run-off.

For example, referring to Fig. 11, a maximum storm of 10 in. corresponds to a seasonal rainfall of 24 in., at San Bernardino and of 33 in. on the Arrowhead water-shed. The total run-off curve for Devil Canyon, in the Arrowhead water-shed (see Fig. 8) gives, for a 33-in. rainfall, a run-off of 7 in. It is concluded, roughly speaking, that the water supply at San Bernardino, resulting from a 24-in. rainfall, should be equal to, if not more than, 7 in.

For the purpose of illustration, there is here presented a determination of the water supply of San Bernardino Valley in the general vicinity of the City of San Bernardino, from the known run-off of the water-sheds of Lytle Creek, Waterman Canyon, and Devil Canyon Creeks. These streams are responsible for the modern valley fill of San Bernardino. They are of granitic and schistose formation and have a dense brush and tree cover. The Valley of San Bernardino is a sloping plain and for the purpose of this illustration is assumed to have a natural brush cover. The characteristics of seasonal rainfall and the analysis of the storms of a wet season for the stations of San Bernardino, Devore, and Squirrel Inn—representing valley, foothill, and mountain water-sheds—are given in Tables 8 and 9.

The determination is made along two distinct lines: First, on the assumption that, for areas comparable in formation and cover, seasons of equal storms will produce in the valley a supply equal to, if not larger than, that in the mountains; and second, on the basis that the relation of seasonal rainfall to the consumptive use of the cover is the same for kindred water-sheds.

Example 1.—The water supply of the valley is to be determined for three seasons in which the maximum storms were 7.5 in., 10 in., and 12.5 in., respectively.

The rainfall-run-off relations of the three mountain water-sheds—Devil Canyon, Waterman Canyon, and Lytle Creek—are shown in Fig. 6. Values for each water-shed are expressed in percentage of the long-period mean rainfall. The curve for Waterman Canyon Creek represents a fair average for the three. It belongs to the Arrowhead water-shed (see Fig. 11) and its mean seasonal rainfall is 32.4 in.

To simplify matters, the Waterman Canyon run-off curve (Fig. 6) is assumed to represent the average run-off conditions, and the Arrowhead water-

TABLE 8.—SEASONAL RAINFALL, IN INCHES.

Season.	San Bernardino Valley floor, Elevation 1 054.	Devore foothills, Elevation 1 990.	Squirrel Inn Mountain water-shed, Elevation 5 200.
1895-96	8.11
1896-97	16.74
1897-98	8.24
1898-99	7.49
1899-1900	8.64
1900-01	17.36
1901-02	11.15
1902-03	17.42
1903-04	9.37
1904-05	20.78
1905-06	19.58
1906-07	23.17
1907-08	15.62
1908-09	17.44
1909-10	15.02
1910-11	16.40
1911-12	13.84
1912-13	11.08
1913-14	21.45
1914-15	19.59	43.57
1915-16	24.72	77.61
1916-17	13.79	41.42
1917-18	13.33	33.33
1918-19	13.62	28.23
1919-20	19.28	37.20
1920-21	16.46	35.21	46.15
1921-22	27.75	59.99	73.58
1922-23	11.04	30.76	41.27
1923-24	11.34	20.32	28.42
1924-25	10.89	26.64	34.81
1925-26	20.40	34.16	45.23
1926-27	20.55	37.24	43.32
Long-period mean	16.15	28.9	42.5

TABLE 9.—ANALYSIS OF STORMS OF A WET YEAR, 1926-27.

Station.	Seasonal rainfall, in inches.	RAIN FALLING IN STORMS OF:		
		Less than 2 in.	2 in. to 6 in.	6 in., or more.
San Bernardino.....	20.55	11.78	8.77*
Devore.....	37.24	3.94* 2.45 2.76
Total at Devore.....	12.34	9.15	15.75*
Squirrel Inn.....	43.32	2.22* 3.25 3.00 3.75 3.25
Total at Squirrel Inn.....	7.55	15.47	20.80*

* One storm.

shed curve to represent average relations between rainfall and maximum storms for the mountain water-sheds. The seasonal rainfall at San Bernardino and its rainfall-storm relations (as shown in Figs. 11 and 12) are representative of the valley.

TABLE 10.—RUN-OFF DETERMINATIONS, EXAMPLE 1.

Location.	Storm, in inches.	Seasonal rainfall, in inches.	Percentage of mean rainfall.	TOTAL RUN-OFF:			
				Class A Years:		Class B Years:	
				In inches.	In percentage of mean rainfall.	In inches.	In percentage of mean rainfall.
(1)	(2)	(3)	(4)	(5)	(6)	(7)	(8)
Waterman	7.5	27.5	85	7.2	8.8
Canyon.....	10.0	33.0	100	10.1	12.7
	12.5	37.0	117	13.0	16.3
San Bernardino	7.5	21.0	130	7.2	45	8.8	54
Valley.....	10.0	24.0	149	10.1	63	12.7	79
	12.5	27.0	174	13.0	80	16.3	100

On Fig. 6 (b), the points marked "San Bernardino Valley" correspond with the values in Column (8) of Table 10. These points might serve as the basis for a water supply curve of the valley for wet years.

*Comparison of Evapo-Transportation Losses.**—In general, it must be assumed that the character and volume of water supply of a country is reflected in the type of its natural vegetation. The water supply of a water-shed is expressed by the average seasonal rainfall or the long-period mean. The average water needs of the vegetation sustained by this mean should bear a more or less fixed relation to the supply.

In a semi-arid country like Southern California, with a pronounced dry and rainy season, and erratic fluctuations of rainfall—not only from day to day, or month to month, or year to year, but also over periods of years—a type of vegetation is produced that is capable of living through a series of comparatively dry years as well as of responding to a wet winter. For a valley area like that at San Bernardino, with a seasonal supply varying from 8 in. to 26 in. and an average of 16 in. (see Fig. 11), the natural vegetation produced would be chaparral, growing to heights of 3 to 4 ft., besides a variety of grasses and herbs, with a consumptive use of 15 to 18 in. At Squirrel Inn the seasonal rainfall averages about 42 in., with a minimum of 26 in. and a maximum of about 80 in. This rainfall, combined with an altitude of more than 5 000 ft., produces chaparral as high as 10 ft. and pine timber requiring for its support a water supply of 25 to 30 in. In both instances the type of vegetation would seem to bear a definite relation to the water supply.

If water-sheds have similar soil formation and are under like climatic conditions, and particularly under the régime of the same cyclonic storms, it is

* In co-operation with A. D. Edmonston, Assoc. M. Am. Soc. C. E., and J. H. Peaslee, Esq.

concluded that the relation of the consumptive use of their cover to their respective water supply is approximately the same, and that for the same season each water-shed will consume about the same percentage of its seasonal supply. The relation of seasonal rainfall to consumptive use is indicated by the rainfall-run-off curve in Fig. 6. If, then, for a series of neighboring water-sheds, both the seasonal rainfall and the total run-off—or its complement, the consumptive use—is expressed in percentage of the long-period mean rainfall, the resulting run-off curves are reduced to a common basis and should not only assume the same shape, but should also fall close together. That this is actually the case for seven water-sheds studied by the writer, is demonstrated by the curves in Fig. 6. The characteristics of these water-sheds are listed in Table 11.

TABLE 11.—CHARACTERISTICS OF WATER-SHEDS.

Name.	Area, in square miles.	ELEVATION.			Mean seasonal rainfall,* in inches.	Description.
		Mini- mum.	Maxi- mum.	Mean.		
San Gabriel River...	222.0	1 000	10 080	4 400	31.0	Steep granitic; some cliffs; flats of old alluvium a minor factor; generally well forested; flashy floods.
San Antonio Creek..	16.9	2 000	10 080	6 600	49.5	Granitic; steep; absorbent detrital slopes; well wooded; flashy floods, but also relatively good summer flow.
Lytle Creek.....	47.7	2 210	10 080	5 500	32.7	South fork, steep and barren, with flashy floods; middle fork, steep and wooded, north fork (the main water supply), absorbent detrital slopes; wooded.
Devil Canyon.....	6.1	1 750	5 250	3 600	32.7	Granitic; steep; some old alluvium flats and detrital slopes; heavy brush cover; alders and sycamores in bed.
Waterman Canyon..	4.5	1 500	5 100	3 600	32.4	Granitic; steep; heavy brush cover; alders and sycamores in bed; no flats, but good detrital slopes at the upper end.
Santa Ana Canyon..	199.0	200	11 500	6 700	37.2	Extensive flats and suspended valleys of ancient alluvium; in the upper water-shed, slopes above flats are broken quartzite and schist which is very absorbent; upper water-shed to the east shows typical desert mountain forest; slopes generally well wooded; extensive meadows.
Mill Creek.....	45.3	300	11 500	6 700	35.8	Steep; granitic; well forested; absorbent broken schist in the upper slopes.

* Estimated from available records for valley stations.

The higher of these water-sheds receives snow above Elevation 5 000 which, in wet years, is a factor that favors late run-off. Many have absorbent detrital slopes which are well wooded.

The close relation of the run-off curves of these water-sheds, both as to alignment and position, particularly those of similar characteristics, is striking and supports the contention that the known rainfall-run-off relation of a water-shed will permit of deductions as to this relation, not only of a neighboring mountain water-shed, but also of a valley floor of kindred formation and cover. In the application to a valley floor, it is to be considered that, for wet years, conditions in the valley favor larger penetration but less run-off. On

the other hand, it is not likely that any water supply will become available for indices of wetness less than 70, because the rainfall of very dry years falls in light storms and is consumed. The character of the cover and its relative consumptive use as well as the over-year effect of the preceding year has an important bearing on the whole question.

Another solution for the determination of the water supply of San Bernardino Valley is outlined in Example 2.

Example 2.—Reference is again made to the run-off curves of Fig. 6, of which the Waterman Canyon curve presents a fair average for indices of wetness greater than 100. The proposed valley water supply curve, therefore, should be located near the Waterman Canyon curve. Equal weight must be given to the points marked "San Bernardino Valley", which were obtained in Example 1 on the basis of the rainfall-storm relations.

For dry years, (Fig. 6 (a)), the position of the curve is governed by the zero point. For ordinary dry years the rainfall at San Bernardino is about 11 in., or about 70% of the mean, with maximum storms of 2.5 to 3.5 in. On the assumption that the valley has a natural brush cover, the zero point has been estimated as follows: For Class A years, at 90% of normal, or 14.5 in.; and for Class B years, 80% of normal, or 12.9 in. The curves have been drawn to follow the general alignment of the mountain run-off curves and marked "Proposed for San Bernardino Valley" (see Fig. 6).

Comparison of Storm Run-Off.—The relation of seasonal rainfall to storm run-off, expressed in percentage of the long-period mean, is shown in Fig. 10. Santa Maria Creek, in San Diego County, has an average seasonal rainfall of 19.4 in., and is characterized by a fringe of low granitic mountains and a comparatively flat valley of a clayey, rather impervious soil.

The data shown on Fig. 6 demonstrate that storm run-off is a minor factor, even for steep water-sheds. As indicated in Fig. 12, the valley floor will receive no storms for seasonal rainfall below normal which would produce surface run-off. The zero point of the valley storm run-off curve, therefore, has been assumed at the 100% point; otherwise, it has been drawn parallel to the general alignment of the mountain storm run-off curves. The water supply resulting from rainfall on the valley floor of San Bernardino Valley, as indicated by the proposed curves in Figs. 6 and 10, is given in Table 12. The methods presented permit of modifications which may be necessary, due to differences in the character of soil and cover between mountain water-shed and valley floor.

DETERMINATION OF RAINFALL PENETRATION FROM TEST WELLS IN PAUBA VALLEY, ON TEMECULA RIVER, RIVERSIDE COUNTY, CALIFORNIA*

Quantitative determinations of deep penetration from rainfall may be made from the rise of the water level in wells. With a water-table relatively close to the surface so that intervening clay strata will not unduly interfere with percolation, the reaction on the water-table from individual storms can be traced distinctly and may serve as a basis for computations of the quantity of recharge.

* In co-operation with B. C. Williams, Assoc. M. Am. Soc. C. E.

Observations to this end, which were made on about fifty test wells covering an area of 400 acres in Pauba Valley, and the analysis of the results, are believed to be of general interest. Incidentally, the volume of return water from irrigation was also determined for the same area.

Pauba Valley is located on the Temecula River at an elevation of about 1 100 ft. above sea level (see Fig. 4). It has a width of $\frac{1}{4}$ mile to 1 mile and a length of more than 5 miles. The average seasonal rainfall is 15.5 in., the minimum 6 in., and the maximum 24 in.

The bottom-lands on which the observations were made consist of a modern alluvium of alternating layers of fine micaceous silt and sand with a top stratum of either silt or sand mixed with humus. The adjoining mesa lands rise about 200 ft. above the valley; they have a gravelly clay loam soil and contribute to the top soil of the bottom-lands.

The following is the log of a test hole in the bottom-lands:

Distance, in feet, below surface.	Material.
0-2	Medium silt loam.
2-4	Sandy loam.
4-6	Heavy silt.
6-7	Fine sand.
7-8	Coarse sand.
8-9	Heavy silt.
9-10	Sand medium.
10-14	Coarse sand.
14	Water-table.

Ground-water stands from 12 to 20 ft. below the surface. The land is planted to alfalfa and is irrigated monthly 4 to 5 in. in depth, for 8 months per year, a record of the water applied being available.

In order to observe the penetration of bottom-lands by rainfall or return water, a large number of 2-in. test wells were driven to depths of 20 ft., perforated at the bottom. These wells are located along lines marked *G, F, H, I*, etc., in Fig. 4, and cover an area of 420 acres. Well observations were made, as a rule, once each week during the period from January, 1926, to March, 1927.

Southern California experienced a series of dry years from the summer of 1922 to April, 1926. An 8.75-in. storm in April, 1926, produced rainfall seepage. In February, 1927, a storm of abnormal magnitude (14.65 in.), occurred which also left a perfect record, while the 3.70-in. storm of December 9, 1926, was a sudden downpour of which a large part ran off.

Representative curves of well observations for 1926-27 are given in Fig. 5, which show the fluctuations of the water-plane along parts of the *N* line (Fig. 4), resulting from rainfall and irrigation. The daily rainfall (accumulative for a storm) and the approximate location of irrigation applications and of the pumping operations at Well No. 30 on the *E* line, about 6 000 ft. up stream from the *N* line, are also shown.

The effect of a storm was determined by measuring the rise in the well some time after the storm when the normal movement (drop or rise) had been re-established. The points of measurement are marked by light, dotted lines in Fig. 5. A porosity factor of 25% was allowed. This would permit some

loss of water by plant life. In some instances the measurements indicated a rise due to run-off, from adjoining mesa lands. On the other hand, there was a compensating run-off away from bottom-lands.

TABLE 12.—WATER SUPPLY FROM RAINFALL IN THE SAN BERNARDINO VALLEY.

Seasonal rainfall, in inches.	Index of wetness.	Available water supply, in inches.	Deep penetration, in inches.	Storm run-off, in inches.	Consumptive use, in inches.
CLASS A YEARS.					
12.9	80	12.90
14.5	90	0	14.50
16.15	100	2.0	2.00	14.15
17.80	110	4.0	3.89	0.11	13.80
19.40	120	5.8	5.56	0.24	13.60
21.0	130	7.25	6.85	0.40	13.75
22.6	140	8.40	7.75	0.65	14.20
24.2	150	9.50	8.60	0.90	14.70
25.8	160	10.50	9.50	1.00	15.30
27.5	170	12.0	10.40	1.60	15.50
CLASS B YEARS.					
12.9	80	12.90
14.50	90	0.82	0.82	13.68
16.15	100	2.10	2.10	14.05
17.80	110	4.35	4.24	0.11	13.45
19.4	120	6.30	6.06	0.24	13.10
21.0	130	8.25	7.85	0.40	12.75
22.6	140	9.70	9.05	0.65	12.90
24.2	150	11.30	10.40	0.90	12.9
25.8	160	12.9	11.90	1.00	12.90
27.5	170	14.50	12.90	1.60	13.00

On the basis of an average rise of the water-plane for the entire area, the curve Fig. 13 (a), was constructed, which shows the relation of intensity of storm to the quantity reaching the water-table. The total effect of the seasonal rainfall in 1926-27 of 23.70 in. on an area of 420 acres, was estimated at 215 acre-ft.

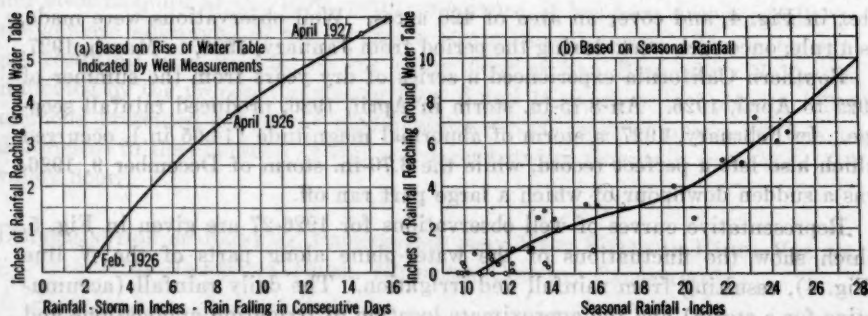


FIG. 13.—PERCOLATION FROM RAINFALL IN PAUBA VALLEY, CALIFORNIA.

The relation of deep penetration to seasonal rainfall was determined by segregating the latter into storms and applying the results to the curve in Fig. 13 (a). By this process the curve in Fig. 13 (b) was obtained. It is based on seasonal rainfall at Aguanga (Elevation 1986), Elsinore (Elevation 1300),

Greenwood (Elevation 1400), and Station C at Pauba Ranch (Elevation 1050), and the fluctuations of Pauba Valley test wells.

The location and effect of irrigation applications is also clearly shown in Fig. 5, and was determined in like manner as that from rainfall, the conclusion being that the irrigation return waters over a large area planted to alfalfa and receiving a seasonal application of 3.5 acre-ft., were 38.4%, or 1.34 acre-ft. per acre.

CONCLUSIONS

For an alluvial valley fill of crystalline origin, the distribution and rate of penetration of moisture from rainfall over large areas are essentially non-uniform and percolation will concentrate in numerous well-defined ducts.

Deep penetration may occur during years of deficient rainfall when the maximum consumptive use of the cover is not satisfied; particularly is this true when the formation is modern.

Rainstorms in Southern California are principally of the cyclonic type and they cover the entire country. Each season is characterized by a storm of outstanding intensity that is chiefly responsible for the water supply which becomes available.

For like seasonal rainfall, the intensity of the maximum storm of a season is greater in the valley than in the mountain water-shed and so is also the resulting water supply.

For like intensity of the maximum seasonal storm, the water supply produced in the valley is approximately the same as that in the mountains (conditions of cover and soil being similar). Given the relation of maximum seasonal storms of mountains and valley, deductions may be made from the known run-off of a mountain area to the available water supply of the adjacent valley floor.

There appears to be a more or less fixed relation between the seasonal rainfall of a water-shed and the consumptive use of its cover, which remains approximately the same for kindred water-sheds of a region. Inasmuch as the consumptive use is only the complement to the run-off, the rainfall-run-off relation of a mountain water-shed will permit of definite conclusions as to the water supply from local rainfall of its adjacent recipient valley area, if these relations are reduced to a common basis and expressed in percentage of the mean rainfall.

Storm run-off of the valley floor is a minor factor, except for compact formations under the effect of the maximum storm of a wet year. A substantial percentage of the rainfall on the valley floor becomes available, varying with the amount of precipitation, the wetness of the preceding season, and the character of soil and cover.

The valley floor in the vicinity of San Bernardino has a mean rainfall of 16.15 in., and a modern alluvial fill of crystalline origin. Assuming a brush cover, the seasonal water supply available from rainfall over a period of years is estimated to average 4.5 to 5 in., or about 250 acre-ft. per sq. mile.

The location and effect of irrigation application is also clearly shown in Fig. 5 and was determined in the manner as that from rainfall, the conclusion being that the irrigation return waters over a large area planted to alfalfa and resulting in a seasonal application of 4.5 acre-ft. were 28.4 acre-ft. more than the 1.84 acre-ft. per acre.

CONCLUSIONS

For an alluvial valley fill of crystalline origin, the distribution and rate of penetration of moisture from rainfall over large areas are essentially non-uniform and percolation will concentrate in numerous well-defined drains.

Deep penetration may occur during years of deficient rainfall when the moisture consumption use of the cover is not satisfied, particularly in the years when the formation is moist.

Fluvial forms in Southern California are principally of the erosive type and they cover the entire country. Each season is characterized by a storm of outstanding intensity that is chiefly responsible for the water supply which becomes available.

For the seasonal rainfall, the intensity of the maximum storm is also the maximum in the valley than in the mountain watershed and so is also the resulting water supply.

For the intensity of the maximum seasonal storm, the water supply produced in the valley is approximately the same as that in the mountain, regardless of cover and soil being similar. Given the relation of maximum seasonal storms of mountain and valley drainage may be made from the known run-off of mountain areas to the available water supply of the adjacent valley floor.

There appears to be a more or less fixed relation between the seasonal rains fall of a watershed and the consumptive use of its cover, which remains approximately the same for kindred watersheds of a region. Inasmuch as the consumptive use is only the complement to the run-off, the rainfall-run-off relation of a mountain watershed will permit definite conclusions as to the water supply from local rainfall of its adjacent recipient valley area. If these relations are reduced to a common basis and expressed in percentage of the mean rainfall.

Storm run-off of the valley floor is a minor factor, except for compact formations, under the effect of the maximum storm of a wet year. A substantial percentage of the rainfall on the valley floor becomes available, varying with the amount of precipitation, the wetness of the preceding season, and the character of soil and cover.

The valley floor in the vicinity of San Bernardino has a mean rainfall of 18.15 in. and a modest alluvial fill of crystalline origin. Assuming a porous cover, the seasonal water supply available from rainfall over a period of years is estimated to average 4.5 to 5 in. of about 200 acre-ft. per sq. mile.

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PRE-DETERMINING THE EXTENT OF A SEWAGE
FIELD IN SEA WATER

By A. M. RAWN,* AND H. K. PALMER,† MEMBERS, AM. SOC. C. E.

SYNOPSIS

This paper presents a method for pre-determining the probable area and extent of a sewage field in sea water, when the quantity of sewage flow and the direction and depth of discharge below the ocean surface are given. The formulas may also be applied to define the probable limits of a sewage field at any given degree of dilution.

Research by the writers fails to reveal any established numerical relation-ship between the known or pre-determined factors surrounding an ocean out-fall outlet and the probable amount of dilution obtained at the ocean surface over the outlet or of the probable spread of the field. In many instances bacteriological surveys made of the area of ocean pollution at sewer outlets have disclosed the extent of the permissible pollution to be far from what the designers anticipated. The predicted area in such cases was based largely upon an "acres-per-second-foot-of-sewage" factor used as a constant rather than as a variable, dependent on the elements of design and conditions existing at the outfall site.

Health standards, as related to shore waters along ocean beaches, define the limits to which pollution of such waters may be carried without creating a menace to health or a public inconvenience. These standards are often found to be maintained by frequent reconstruction of the ocean outlet following quarantine of the adjacent shore waters. By means of the formula presented herewith, an attempt is made to eliminate some of the elements of doubt and conjecture regarding the probable spread of a sewage field and to aid in the design of ocean outlets which will not of necessity be abandoned or recon-structed prior to the expiration of their anticipated useful life.

NOTE.—Written discussion on this paper will be closed in September, 1929.

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The particular factor that influenced the undertaking of this series of experiments is the contemplated construction of an ocean outfall by the Los Angeles County Sanitation Districts on the California coast line, south of the City of Los Angeles. The information and data assembled during the series of experiments, and the formulas derived from the experimental and research work, are presented for the information of those engaged in similar work or studies.

NOTATION

The following notation has been used in the paper:

- a = constant in the equation of the path from a horizontal outlet.
- A = area, in square feet, of the cross-section of the rising column of water at any level (y) under observation.
- B = constant.
- D = diameter of the outlet, in inches.
- E = horizontal distance from the center of the rising column to the edge of the sewage field.
- F = constant used in determining velocity in the sewage field.
- K = constant.
- L = length of path between outlet and surface.
- n = number of outlets.
- P = thickness of sewage field.
- Q = quantity of sewage or fresh water.
- S = dilution factor.
- S_0 = dilution factor at head of rising column.
- t = time, in seconds.
- U = velocity of ocean current.
- V = velocity of the fresh-water jet, or of diluted sewage in the sewage field.
- x = horizontal distance from the outlet to the center of the rising column at any level, y , under consideration; or the horizontal distance from the center of the rising column at the surface to any point in the field.
- X_0 = radius of virtual ring from which the sewage field spreads.
- X = horizontal distance from the outlet to the center of the rising column at the surface (see Fig. 1).
- y = vertical distance from the outlet to the point in the center of the rising column that is under consideration.
- Y = total depth of salt water above the outlet.

EARLY OBSERVATIONS OF THE BEHAVIOR OF A FRESH-WATER JET IN SALT WATER

From preliminary studies and minor experiments with small colored fresh-water jets about $\frac{1}{4}$ in. in diameter, discharging less than $\frac{1}{4}$ gal. per min. in clear sea water, the following behavior was indicated. When discharged vertically

upward from a submerged outlet a jet was seen to rise very much in the form of a dense cloud of smoke; it rolled and billowed in much the same manner, taking finally a shape which approximated an inverted cone with its base resting at the surface of the water and its apex at the point of discharge. Repeated trials indicated that the cone was uniformly constant in dimension and appearance for like conditions of discharge.

From a jet of fresh water discharged horizontally below the surface of the salt water the path of the fresh water appeared to approximate that of a projectile, except that it curved upward and was retarded by the turmoil occasioned by the billowing of the fresh water in the same manner as smoke billowing from a stack. A nozzle was fixed at a depth such that, when the jet was discharged vertically upward, it just reached and broke the surface of the salt water. The writers demonstrated that the same quantity of water from the same nozzle, discharged horizontally at the same depth would not reach and break the surface of the salt water as a well-defined jet. This indicated that, other conditions being equal, the absorption of sea water into the sewage—or the degree of intermingling of the two waters—was greater for a horizontal than for a vertical discharge outlet.

With the jet pointing vertically downward, the path of fresh water gave the appearance of an inverted fountain with the head fluctuating or varying slightly in elevation. The fresh water traveled, sometimes, all to one side of the jet and frequently, in its upward path, completely surrounded the downward jet. The upward path then took the same approximate form as that from a vertical discharge line, except that it was not quite so concentrated in color and for a like amount of discharge presented a somewhat larger ring as it broke the surface. For a jet inclined 45° upward, conditions appeared to be about an average between that for a vertical and a horizontal discharge.

A cap in the form of a funnel with the spout closed, was lowered over the vertical jet until the edge of the funnel was below the point of outlet. The fresh water then gave the appearance of passing over an inverted weir, the upward path from the edges of the weir being about the same form as the rising column from a vertical jet, but with greater diffusion of fresh water into salt water and a larger field where the fresh water broke the surface of the salt water.

The observation of the behavior of the streams of fresh water from the miniature jets in the salt water formed the basis for the design of an apparatus to be used in the subsequent studies. It was determined from these meager observations that it would be necessary to be able to measure actual dilution at the surface, as produced under varying conditions of depth, quantity, velocity, and direction of discharge.

DESCRIPTION OF METHODS AND APPLIANCES

The first observations were made in tubs of salt water and in the quiet lagoons at Alamitos Bay, California. They indicated little that was conclusive because no method was provided for measuring the value of the dilution factor at the time the stream reached the surface of the salt water or during its spread over that surface.

The site chosen for carrying the experiments to a conclusion was a basin in Los Angeles Harbor about 2 acres in extent, protected on two sides by land and, with the exception of a small entrance, on the other two sides by fairly tight sheeted wharves. Few steamers passed the entrance, and the only other disturbance due to wave action was occasioned by fishing boats and yachts brought into the basin for repairs. Equipment was mounted on a large raft moored in the basin which permitted the experiments to be made without reference to the tides. Depths of 13 ft. were obtained at high tide with 9 ft. possible at all times. The arrangement is shown in Fig. 1.

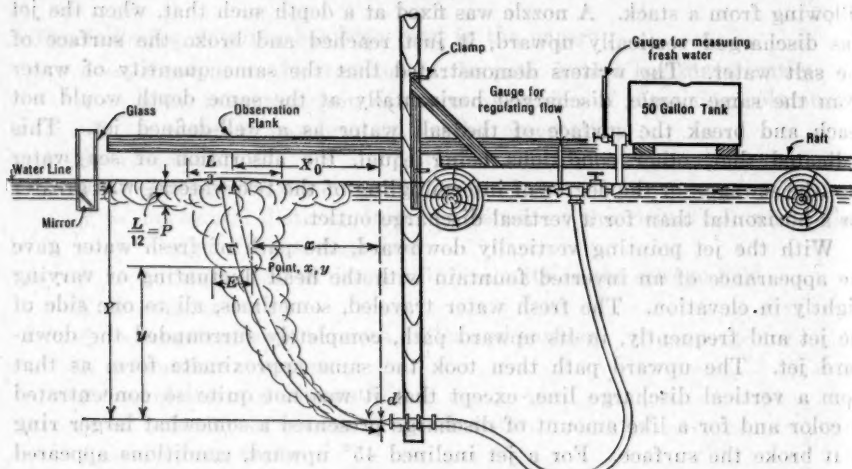


FIG. 1.—EXPERIMENTAL APPARATUS.

A fresh-water tank (a 50-gal. oil drum), was mounted on its side on the raft; the contents could be discharged through a $1\frac{1}{2}$ -in. flexible hose, 12 ft. long. Pipe couplings, nipples, and reducers, could be attached to the end of the hose, permitting discharge through varying sizes of nozzles, which were ordinary pipe nipples ranging from $\frac{1}{4}$ in. to $1\frac{1}{4}$ in. in inside diameter. A few experiments were performed with $1\frac{1}{2}$ -in. and 2-in. nozzles, with a section of $2\frac{1}{2}$ -in. fire hose as a discharge line. The permanent end coupling on the hose was attached to a 1-in. by 6-in. board, 16 ft. long, which could be clamped to an upright post on the raft with the nozzle at any desired depth.

Flow from the drum was controlled by a globe valve located about a foot below the lowest elevation of the drum. Thus, the variation in flow due to change in head, as the drum emptied, was minimized. As a check on the uniformity of flow a gauge glass was placed in the line below the valve, and was kept, as nearly as possible, at a constant head by adjusting the valve opening from the drum.

The drum was carefully calibrated and the rate of flow during any one experiment was determined by measuring the quantity of water discharged during that experiment. Time was measured to the nearest second and the

fluid surface drop in the tank was indicated by the fall in a gauge glass in the outlet just above the globe valve.

Eosine Y was introduced into the fresh water as a coloring agency, the proportions being about $\frac{1}{4}$ grain per gal. for shallow-water experiments when resulting low values of S were to be measured and as much as 2 grains per gal. for either deeper experiments or those in which the field spread was to be measured.

The value of S at any time was measured by diluting with sea water a measured sample of the tank fluid until it compared in color with the sample taken from the sewage field. Thus, if 10 cu. cm. of the fluid taken from the tank were diluted until it matched in color a sample taken from the jet or field and the sample was then found to contain 120 cu. cm. of combined colored fresh water and sea water, the resulting S was $\frac{120}{10} = 12$.

Samples were compared in 8-oz. oil-sample bottles, $1\frac{1}{2}$ in. in diameter and 6 in. long, and the high dilutions were matched by holding the bottles, base downward, over a mirror so as to get a maximum concentration of color by looking indirectly through the bottle along its long axis. It was found that differences in dilution could be measured in this manner to a value of 150 for S with a probable error of about 4 per cent.

Conditions below the surface, particularly those which marked the plane between the clear salt-water and the colored fresh-water field, were investigated by right-angle reflection. The apparatus consisted of an upright oblong box closed at the bottom and open at the top. One side of the box was fitted with a glass pane, and a mirror was set in the bottom at an angle of 45° with the glass side. The possibilities of such an instrument are apparent.

The paths of the smaller streams of fresh water were found to be subject to some variation as a result of slight currents in the sea water and in order to measure the horizontal co-ordinate of the path of a horizontal jet with a minimum of possible error, a double jet was placed at times on the nozzle. It consisted of a "tee" and the two jets discharged horizontally in opposite directions. The distance between the centers of the two rising columns was measured at the surface; from this was subtracted the distance between the two nozzle ends and the remainder was divided by 2, giving the required distance without reference to variation in the path which was the same for both jets. Interference between two rising columns below the surface and the spread of a double field was investigated with the same apparatus slightly modified.

The spread of the field on the surface was determined by observing the time when the edge of the field passed fixed points along the edge of the raft and along a plank cantilevered over the water at right angles to that edge.

As a check on results obtained from the foregoing experiments and in order to fix with greater accuracy the upper limits of the curves, values of S were determined for four known outlets in the vicinity of Los Angeles,

namely, those at Los Angeles Hyperion Outfall, Long Beach, Santa Monica, and Santa Barbara. One at Catalina Island was investigated, but the sewage was found to be so diluted on reaching the water surface that samples taken would not yield results.

In the existing outfall investigations the salt content of each sample was measured. A solution of silver nitrate was prepared and standardized by titrating 2 cu. cm. of ocean water with it; 2 cu. cm. of each of the samples was then titrated in the same manner and when there was sufficient salt in the raw sewage to effect results, it also was titrated. By this means values of S up to 35 or 40 could be determined with accuracy.

To determine the thickness of a sewage field the difference in electrical conductivity of sea water and diluted sewage was noted with a Wheatstone bridge. The galvanometer was adjusted with the electrodes on both arms of the bridge immersed in clear salt water well below the layer of sewage. On raising one arm of the bridge the galvanometer would give a kick as the electrodes entered the diluted sewage. A dilution of 25 could be detected in this way.

VERTICAL JETS

It was apparent that there were fewer factors affecting the dilution from vertical jets than from other types, so they were studied first. A series of 222 observations was made at depths ranging from 1.2 to 10.5 ft. and using $\frac{3}{8}$ -in., $\frac{1}{2}$ -in., $\frac{3}{4}$ -in., 1-in., and $1\frac{1}{4}$ -in. nozzles. The number of experiments at various depths for each size of outlet are shown in Table 1.

TABLE 1.—NUMBER OF OBSERVATIONS ON VERTICAL JETS.

Depth, in feet.	DIAMETER OF NOZZLE OPENING, IN INCHES.					Total number of observations.
	$\frac{3}{8}$.	$\frac{1}{2}$.	$\frac{3}{4}$.	1.	$1\frac{1}{4}$.	
1.2	...	3	3	3	3	12
1.5	10	3	7	3	10	33
2.5	3	3	3	3	3	15
3.5	3	3	3	3	3	15
4.5	4	3	3	3	3	16
5.5	11	3	17	3	14	48
6.5	3	3	3	3	3	15
7.5	3	3	3	4	3	16
8.5	...	3	3	3	3	12
9.5	7	...	11	3	16	37
10.5	3	...	3
Total.....	44	27	56	34	61	222

If for any fixed depth the results of each experiment are plotted with values of Q , in gallons per minute, as ordinates and S_0 as abscissas the equation of the curve takes the form:

$$S_0 Q^{0.67} = f(y) \dots \dots \dots (1)$$

and this constant is the same for all sizes of nozzles. Subsequent comparison with results at the existing Long Beach Outfall while 3 000 gal. per min. of

sewage was being discharged at a depth of 9.5 ft. and where the resulting S_0 was small, showed that the equation took the form:

$$(S_0 - 1) Q^{0.61} = f(y) \dots \dots \dots (2)$$

The factor $(S_0 - 1)$ is used instead of the factor, S_0 , in order to avoid the anomalous case arising from the discharge of a large quantity of sewage at shallow depth, under which condition the value of S_0 might become less than unity, which would correspond to a concentration of the sewage. Since $(S_0 - 1)$ indicates the number of units of sea water per unit of sewage, there is a logical reason for its use.

In Fig. 2 results are plotted at the three depths at which the major number of experiments were taken, and the result of the experiment described at Long Beach is also shown.

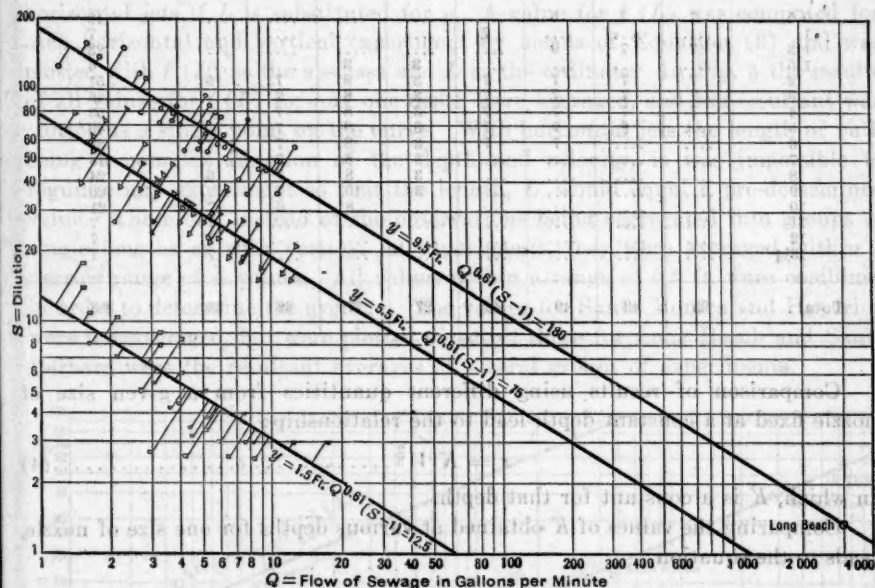


FIG. 2.—RELATION BETWEEN DILUTION AND QUANTITY AT CONSTANT DEPTHS.

HORIZONTAL JETS

This type of discharge is complicated by the fact that the rising column follows a curve similar to the path of a projectile, except that it curved upward. The direction of the path is modified by the inertia of the salt water picked up by the rising column, the tendency being to reduce its horizontal component. A total of 388 experiments was made, using horizontal jets, the numbers at each depth, with different sizes of nozzles, being shown in Table 2.

It was apparent from the beginning that the value of S_0 was greater from horizontal jets than from the vertical, other conditions being equal, and this group of experiments indicated that for this case Equation (1) should be written:

$$(S_0 - 1) Q^{0.61} = f(L) \dots \dots \dots (3)$$

The first requisite in the solution of Equation (3) was to determine the length of the rising column between the point of discharge and the salt-water surface. This length is, in turn, dependent on the equation of the curve of the path of the rising column. In order to avoid, as far as possible, error introduced by lateral currents in the sea water, the horizontal co-ordinate, X , was measured with the use of the double nozzle heretofore described.

TABLE 2.—NUMBER OF OBSERVATIONS ON HORIZONTAL JETS.

Depth, in feet.	DIAMETER OF NOZZLE OPENINGS, IN INCHES.								Total number of observations.
	$\frac{1}{4}$.	$\frac{3}{8}$.	$\frac{1}{2}$.	$\frac{3}{4}$.	1.	$1\frac{1}{4}$.	$1\frac{3}{4}$.	2.	
0.5	7	7	5	19
1.	7	7
1.3	7	7	9	23
1.5	8	3	6
2	7	7	7	7	3	8	7	5	51
3	7	7	7	7	3	8	7	...	46
4	7	7	7	7	3	8	7	7	53
5	...	8	7	7	3	8	7	...	40
6	7	7	3	8	7	7	39
7	3	8	7	...	18
8	3	8	7	6	24
9	3	10	7	...	20
10	8	6	7	21
11	7	7
12	7	7
13 $\frac{1}{4}$	7	7
Total	35	43	49	42	27	77	62	53	388

Comparison of results using different quantities from a given size of nozzle fixed at a constant depth lead to the relationship:

$$x = K V^{\frac{2}{3}} \dots \dots \dots (4)$$

in which, K is a constant for that depth.

Comparing the values of K obtained at various depths for one size of nozzle, leads to the equation:

$$K = B \sqrt[3]{y} \dots \dots \dots (5)$$

in which, B is a constant for that size.

Furthermore, comparing the values of B for nozzles of different sizes, indicates that it has a progressive increase expressed in the equation:

$$B = \sqrt[3]{D} \dots \dots \dots (6)$$

Combining Equations (4), (5), and (6), and substituting for $\sqrt[3]{V^2 D}$, the constant, a , Equation (4) becomes,

$$x = a \sqrt[3]{y} \dots \dots \dots (7)$$

which is a cubical parabola with the origin at the nozzle.

The formula for the length of this path is too cumbersome to use in practice, but the two following approximations will give the length with sufficient accuracy:

Case 1, when $\frac{Y}{X} > \frac{1}{3}$,

$$L = Y + (0.8a) < \frac{3}{2} - \frac{0.1685 a^2}{\sqrt[3]{y}} \dots\dots\dots(8)$$

Case 2, when $\frac{Y}{X} < \frac{1}{3}$,

$$L = X \left\{ 1 + 0.9 \left(\frac{Y}{X} \right)^2 \right\} \dots\dots\dots(9)$$

The exact formula and the two approximations (Equations (8) and (9)) were developed by Mr. Gordon Ward, of the California Institute of Technology.

RELATION OF DILUTION, S_0 , TO LENGTH OF PATH, L

Equation (2) for vertical jets becomes identical to Equation (3) for horizontal jets if L is substituted for y . A value for $f(L)$ was computed for each horizontal and vertical experiment by means of Equation (3) and was plotted with $f(L)$ as the abscissa and L as the ordinate. In Fig. 3 the results of all values for $f(L)$ for any one depth were averaged, and the resultant was plotted as a single point on the curve. With horizontal jets the length of path being a complex function of the depth and velocity, it was impossible to regulate any experiment so that the length, L , would equal a pre-determined value. Therefore, instead of the observations being segregated into groups of single lengths as with vertical jet experiments, they were averaged within a narrow range of L values. All values within a range of 0.5 ft. were combined in order to determine the average. The values for Santa Monica and Hyperion were not averaged, but were plotted directly; those for Long Beach and Santa Barbara were the resultant averages of several groups of experiments.

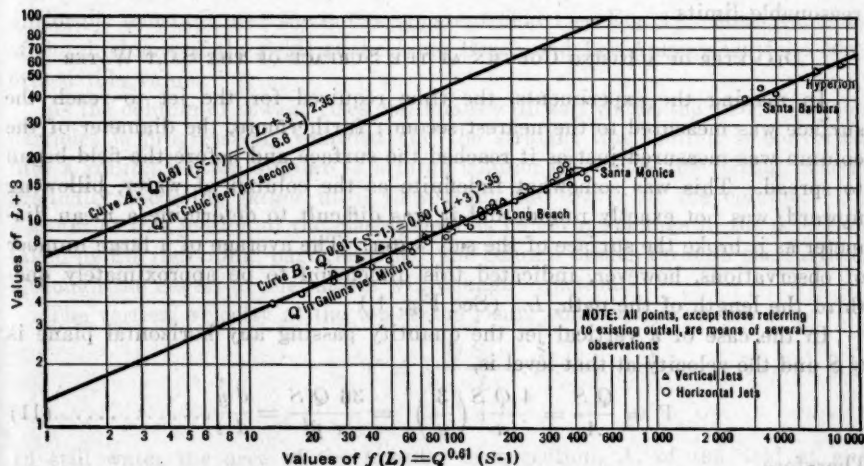


FIG. 3.—RELATION BETWEEN DILUTION AND LENGTH OF PATH.

When first plotted, the results lay on a curve, and it was necessary to add 3 ft. to the length of path, which resulted in the equation,

$$(S_0 - 1) Q^{0.61} = 0.5 (L + 3)^{2.35} \dots\dots\dots(10)$$

in which, Q is measured in gallons per minute. For large discharges it is more convenient to measure Q in cubic feet per second in which case the coefficient may be readily changed.

JETS DISCHARGING VERTICALLY DOWNWARD

In some cases outfall pipes are carried on wharves and the pipe is clamped to a pile so as to discharge the jet downward some distance below the surface of the water. The fresh water then acts in a manner similar to an inverted fountain and the distance to which the water flows downward before turning up, is approximately $0.5 V$. This distance is only approximate because the so-called inverted fountain is very unstable, the "head" constantly descending and rising. At Santa Monica, where this system of discharge is used, it was noted that the sewage came to the surface in waves, the flow at times almost ceasing, followed by a "crest" when a large quantity would boil up.

The resulting dilution is apparently that due to the total length of path from the end of the outlet down to the turning point and back to the surface, but in fairly large outlets it is probable that little dilution occurs on the downward path, so that the effective length would be only from the bottom of the downward jet to the surface.

JETS DISCHARGING UPWARD AT 45 DEGREES

Experiments were conducted with jets discharging 45° from the vertical and the results indicated that the length of path, L , was the mean of the lengths for vertical and horizontal jets, other conditions being equal. No formula is given for this case. However, it appears that the mean of vertical and horizontal formulas will indicate the value for S_0 for inclined jets within reasonable limits.

DIAMETER OF A RISING COLUMN AT THE SURFACE OF THE SALT WATER

In making the experiments, the time required for the jet to reach the surface was measured to the nearest second; furthermore, the diameter of the column was measured just as it reached the surface and before the field began to spread. This was somewhat indefinite as the column of water, billowing upward, was not exactly round, and it was difficult to determine a mean diameter as it broke the surface of the salt water. The average of a large number of observations, however, indicated this dimension to be approximately one-third the length of the path, L . (See Fig. 1.)

In the case of a vertical jet the quantity passing any horizontal plane is QS and the velocity at that level is,

$$V = \frac{QS}{A} = \frac{4QS}{\pi} \left(\frac{3}{y} \right)^2 = \frac{36QS}{\pi y^2} = \frac{dy}{dt} \dots \dots \dots (11)$$

Therefore,

$$t = \int_0^y \frac{\pi y^2}{36QS} dy \dots \dots \dots (12)$$

Since S is a complex function of y , this integration (Equation (12)) was performed mechanically. A comparison of the observed and calculated time of

rising demonstrated that the observed time was approximately 0.6 of the calculated time, indicating that the assumed area, $A = \frac{\pi}{4} \left(\frac{y}{3} \right)^2$ was too great and that the virtual diameter of the column was more nearly $\frac{y}{4}$, for vertical jets, or $\frac{L}{4}$ for horizontal jets.

By virtual diameter is meant the diameter of a theoretical column with uniform velocity at all points in any given cross-section, and of such size that the time of rise will be equal to the observed time. In actual cases the outer parts of the column contain rolling water with an upward velocity relatively slow as compared with the center of the column.

For large sewers the diameter of the nozzle should be added to $\frac{y}{4}$ or $\frac{L}{4}$ to obtain the true diameter of the column. The importance of this added factor varies inversely with the depth of discharge below the water surface.

THICKNESS OF THE SEWAGE FIELD

As the dilute sewage in the rising column reaches and breaks the surface of the salt water its tendency is to spread laterally in all directions over the heavier salt water. The plane of contact between the salt water and the lighter mixture of salt and fresh water is not clearly defined, owing to the natural tendency of the two fluids to intermingle. The average of many observations indicates a value of $\frac{L}{12}$ for the field thickness and that this thickness remains uniformly constant over the entire apparent field (see Fig. 1). Measurements of the thickness of the fields at the Long Beach and Los Angeles Outfalls confirmed this value.

As the column of sewage rises and mixes with salt water, the velocity of the particles is progressively retarded and the straight line motion is converted into a rolling motion similar to a large column of smoke, the rolling masses gradually becoming larger until they reach the center of the column. On account of this motion of the masses there is no abrupt change in the direction taken by the individual particles where the column breaks through the surface, although the center of the masses will change direction.

The vertical velocity at the top of the rising column is,

$$V = \frac{Q S_0}{\pi \left(\frac{L}{4} \right)^2} \dots \dots \dots (13)$$

In still water the area of the annular cross-section, A , of the field at any distance less than E from the center of the column (see Fig. 1), will be represented by the formula,

$$A = 2 \pi x P \dots \dots \dots (14)$$

The virtual diameter at the top of the rising column is $2 X_0 = \frac{L}{4}$, and the lateral velocity at the head of the rising column will be represented by,

$$V = \frac{Q S_0}{\pi P \left(\frac{L}{4}\right)} \quad (15)$$

Assuming that no loss of energy occurs in the change from vertical movement in the rising column to horizontal movement as the field spreads, by Equation (13) and Equation (15),

$$\frac{Q S_0}{\pi \left(\frac{L}{4}\right)^2} = \frac{Q S_0}{\pi P \left(\frac{L}{4}\right)}$$

$$\text{or, } P = \frac{L}{16}$$

This indicates that the observed velocity of the horizontal motion in the field spread is less than the theoretical value, the latter being determined by assuming no loss in energy as vertical motion is changed to horizontal. Increased turbulence at the column head indicates energy loss at that point, and it is believed that the ratio, 12 : 16, may be fairly representative of such loss. Therefore, the value for P is assumed to be (see Fig. 1),

$$P = \frac{L}{12} \quad (16)$$

SIZE OF THE SEWAGE FIELD

As the fresh water spreads the dilution continues, but since the turbulence is much less than in the rising column the rate of dilution diminishes correspondingly. It was observed that the colored fresh water moved over the salt water in concentric waves, each wave being followed by an interval of apparently clear salt water. These waves continued for some distance from the head of the rising column, and gradually merged to form a field of uniform color. This fact is noted to illustrate the difficulty of securing a single representative sample. Those taken from the same location in the field at different times, other conditions being equal, indicated that the dilution remained constant, within the limits of error of observation, at that location.

Experiments made with a view to obtaining the progressive increase in dilution from the head of the rising column to the edge of the apparent field, did not yield results that were as consistent as those indicating the dilution over the rising columns. This was probably due to the manner in which the sewage left the column head in waves. However, such experiments did show progressive increase in dilution as the distance from the head increased, and the time required for the sewage to travel a given distance was exceedingly uniform for like conditions. The latter was found to be expressed by the equation,

$$t = \frac{2}{3 F} x^3 \quad (17)$$

Differentiating Equation (17),

$$\frac{dx}{dt} = \frac{F}{\sqrt{x}} = V \dots \dots \dots (18)$$

The quantity of diluted fresh water passing a ring is QS and the area of such annular cross-section, by Equation (14), is $2\pi Px$, thus the velocity is,

$$V = \frac{QS}{2\pi Px} \dots \dots \dots (19)$$

Equating this value to Equation (18) and transposing terms:

$$S = \frac{2\pi PF\sqrt{x}}{Q} \dots \dots \dots (20)$$

At the head of the column from which the field expands, $x = X_0$ and $S = S_0$; substituting these values in Equation (20),

$$F = \frac{QS_0}{2\pi P\sqrt{X_0}} \dots \dots \dots (21)$$

and substituting this value for F in Equation (20),

$$S = S_0 \sqrt{\frac{x}{X_0}} \dots \dots \dots (22)$$

Observations of existing fields and a number of experiments indicate that $X_0 < \frac{L}{8} + \frac{D}{24}$. However, the use of this value,

$$X_0 = \frac{L}{8} + \frac{D}{24} \dots \dots \dots (23)$$

in Equation (22) is recommended.

In a confined channel with approximately parallel sides the cross-section of the field can increase only by thickening. The kinetic energy cannot increase and, therefore, as the velocity decreases with the increasing dilution, it is evident that the field must thicken, and, as soon as the thickness equals the depth of the water in the channel, no further dilution is possible. It is evident, therefore, that a constricted channel makes a poor location for a sewer outlet, and where it is the only available location the initial dilution should be as high as possible.

Some currents are nearly always present in the ocean, and in order to consider their effect on the field it is necessary to consider the velocity of spreading. The two particularly important considerations are the distance the field will travel directly against an ocean current and the distance it will travel directly with the current, until a certain value of S is attained.

The maximum distance the sewage will travel against the current can be expressed as,

$$e = x - x' = x - Ut \dots \dots \dots (24)$$

in which, x' equals the distance the ocean current will travel in time, t , at a velocity, U , and e is the maximum distance from the center of the column head

to which the sewage can travel against the ocean current. Under conditions expressed in Equation (24), $V = U$. Then, from Equation (18), $U = \frac{F}{\sqrt{x}}$, and,

$$x = \left(\frac{F}{U}\right)^3 \dots \dots \dots (25)$$

From Equation (17),

$$U t = \frac{2}{3} \left(\frac{F}{U}\right)^2 = x' \dots \dots \dots (26)$$

Substituting the values of x and x' from Equations (25) and (26) in Equation (24),

$$e = \left(\frac{F}{U}\right)^2 - \frac{2}{3} \left(\frac{F}{U}\right)^2 = \frac{1}{3} \left(\frac{F}{U}\right)^2 \dots \dots \dots (27)$$

in which, the value of e is independent of the value of S .

The distance the sewage will travel with the current will be expressed by the relation,

$$e = x + U t \dots \dots \dots (28)$$

MUTUAL INTERFERENCE FROM MULTIPLE OUTLETS

The theory has been advanced in the past, and incorporated into the actual design and construction of numerous ocean outlets, that, other conditions being equal, the use of multiple outlets is advantageous.

An inspection of Equation (10) for S_0 would indicate that such an assumption is true, because the value of S_0 for $\frac{Q}{n}$, for a given depth, is greater than the value of S_0 for Q and the distance that sewage must travel to attain a definite value for S is a function of $\left(\frac{S}{S_0}\right)^2$.

Interference Between Rising Columns Below the Salt-Water Surface.—Two vertical outlets were attached to the discharge hose so as to permit simultaneous discharge from both at equal depth and at constant distance between centers of rising columns. As long as the depth of discharge below the surface was less than three times the distance between the two outlets, the observed and calculated values of S_0 were the same. However, when the depth of discharge below the surface exceeded three times the distance between the two outlets, the two rising columns interfered below the salt-water surface, decreasing the area of the columns in contact with pure salt water. This resulted in the observed value for S_0 being less than the theoretical for each column alone. At extreme depths, where the ratio of depth to distance between centers was great, the observed values approached the theoretical value for one outlet discharging the combined flow of the two.

A series of experiments was performed in which two horizontal jets were directed toward each other. The resulting observed values of S_0 in this instance approached very closely those for the foregoing experiment with two vertical jets at equal depths. This is reasonably explained by the fact that

the two horizontal velocities directed toward each other were neutralized, the path of each was shortened, and the two columns mingled shortly after discharge and rose together as a single vertical column.

Six slots, 1 by 1-in., were cut in the rim of a can, which was inverted and placed over a vertical nozzle. This divided the flow from the nozzle into six small columns, and at shallow depths, where these columns did not interfere in rising, the resulting observed value for S_0 was that due to each small stream alone; but after interference became effective, at greater depths, the advantage was nullified.

Under the circumstances, very little horizontal velocity was imparted to the fresh water flowing through the orifices, the fresh water escaping as if it were flowing over inverted weirs with an acceleration due to the difference of the specific gravity of the two fluids and equal to $\frac{g}{40} = 0.8$.

A similar experiment was performed by inverting a can over a vertical nozzle, except that, in this instance, no slots were cut into the edge of the can. Under these circumstances the fresh water rose in the form of a hollow column, all the dilution taking place on the outer surface thereof. As the depth of discharge was increased the ratio of the observed value of S_0 with the diffuser, to the calculated value without the diffuser, decreased from 1.5 at a depth of 2 ft. to 1.3 at 6 ft. and 1.08 at 10 ft.

Merging of Fields.—Ordinarily, when multiple outlets are used, the fields will merge a comparatively short distance from the rising columns. The merging of the individual fields may result in a thickening of the major field, an increased lateral velocity of spread, or a combination of both. The following data indicate that the increased lateral velocity predominates, although there is probably some thickening of the major field.

In calm water, with no currents, the major field can be considered as a number of individual fields each roughly corresponding in shape to a sector of the circular area of the major field with the column head near the apex. The condition obtaining in each individual field, under such a circumstance, was roughly reproduced by discharging a jet from a horizontal nozzle close to the surface, thereby imparting to the field a horizontal velocity in the direction in which the field would normally expand if its natural spread were laterally obstructed by other forces.

A tabulation of the results obtained from this experiment indicate that X_0 is about twice the X_0 value obtained when the field was permitted to expand equally in all directions.

In Equation (22), doubling the value of X_0 likewise doubles the value of x if $\left(\frac{S}{S_0}\right)$ remains constant. In the foregoing experiment the field was roughly circular and x represented the diameter. Under normal conditions of spread, without currents present, x will represent the radius. It follows therefore, that since the diameter of the field in the experiment was about twice the radius of the normal field the two are roughly equivalent.

This leads to the conclusion that even if the individual fields were still more distorted, approaching the shape of circular sectors, the assumption of equivalent areas would still hold. Results of investigations at the Hyperion Outfall indicate that to be the case.

The area of the major field, from such conclusions, will be n times the area of each individual field and its radius, or the distance the sewage will travel in calm water is obtained from the value of x in Equation (22), thus,

$$x = X_0 \left(\frac{S}{S_0} \right)^2 \sqrt{n} \dots \dots \dots (29)$$

Due to the increased velocity of spread in the major field the time for Q to reach the limit, E , of the major field is the same as for $\frac{Q}{n}$ to reach the limit of its individual field.

Let the total flow through all the outlets be equal to,

$$Q' = n Q \dots \dots \dots (30)$$

$$X_0' = X_0 \sqrt{n} \dots \dots \dots (31)$$

$$F' = \frac{Q' S_0}{2 \pi P \sqrt{X_0'}} \dots \dots \dots (32)$$

Then, substituting Equations (30) and (31) into Equation (32):

$$F' = \frac{n Q S_0}{2 \pi P \sqrt{X_0' \sqrt{n}}} = n^{\frac{1}{2}} \frac{Q S_0}{2 \pi P \sqrt{X_0'}} = n^{\frac{1}{2}} F \dots \dots \dots (33)$$

From Equation (17), the time required to reach the outer limit of the combined field is equal to:

$$t' = \frac{2}{3 F'} (x')^{\frac{3}{2}} \dots \dots \dots (34)$$

and substituting F' in Equation (33):

$$t' = \frac{2}{3 n^{\frac{1}{2}} F} x^{\frac{3}{2}} n^{\frac{1}{2}} = \frac{2}{3 F} x^{\frac{3}{2}} = t \dots \dots \dots (35)$$

STRATIFICATION

It has been found that when water is undisturbed by wind or wave action, a difference of 2.5° Fahr.* in its temperature is sufficient to cause stratification. The equivalent of this difference is shown in Fig. 4, in which temperature of sea water is plotted against the specific gravity of the sewage for different values of S at 60° Fahr. From this, it may be seen that raising the temperature of sea water from 60° to 62.5° Fahr., reduces the specific gravity from 1.02540 to 1.02479, the latter being equivalent to the specific gravity of sewage at 60° Fahr., diluted to $S = 105$.

This indicates that if S is less than 105 the mixture will tend to come to the surface. Actual experiment shows, however, that mixtures with a much higher value of S will come to the surface, probably assisted by the inertia imparted by the initial velocity.

* "Solving Sewage Problems," by Fuller and McClintock, p. 157.

After reaching the surface the lighter fluid spreads laterally over the sea water and its tendency to remain stratified is exceedingly persistent. Oceanographic evidences of stratification in clear sea water are common in places where only slight changes in specific gravity are apparent. Observations have disclosed that stratification temporarily exists where the heavier water forms the upper stratum, the layers thus being in a state of unstable equilibrium. They remain so until disturbed sufficiently to cause a re-adjustment to stability with the heavier water at its proper level.

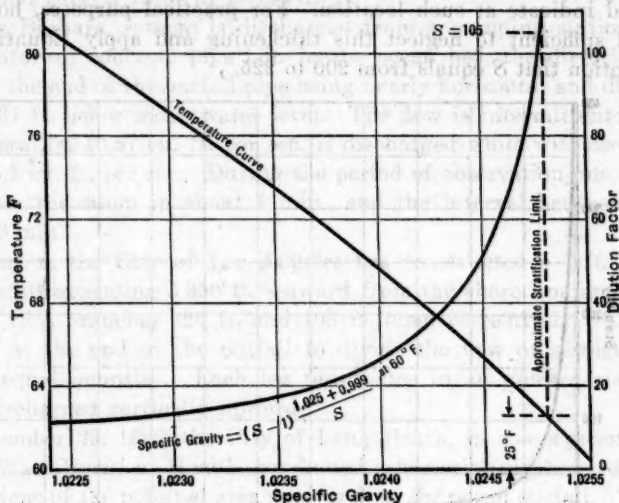


FIG. 4.

It is concluded, therefore, that the sewage field may not be expected to thicken before it has traveled such a distance from the column head as is needed to obtain a value in excess of 100 for S . The probable tendency will be for the sewage to remain distinctly stratified until a value of $S = 200$, or more, is reached; at which time (see Fig. 5), the specific gravities of the sea water and the dilute sewage are so nearly alike as to tend to merge, causing field thickening and consequent great dilution.

Wind and wave action or other disturbance will hasten the mingling of the thin sheet of sewage with the sea water and will have a pronounced effect on the breaking up of stratification; therefore, the most pronounced apparent fields will be observed during the periods of greatest calm.

In bacteriological surveys conducted about the Los Angeles City Outfall at Hyperion, a presumptive count of not to exceed 10 *B. Coli* per cu. cm. of sample taken is considered as marking the edge of the polluted area. Applying Equation (22) to the actual sewage field as found at Hyperion indicates that the value of S at the outer limit of the field is 212. Time as well as dilution may well be responsible for the bacterial destruction and it is altogether probable that the sea water exercises a bactericidal effect on the sewage. It must be apparent that a presumptive count of 60 000 \pm *B. Coli* per cu. cm. of sewage cannot be reduced to 10 per cu. cm. of sewage and sea water by dilution alone

in so short a distance from the column head as is usually found in bacteriological surveys.

It is undoubtedly true that, after the breaking up of stratification, the field thickens materially, reducing the lateral velocity, increasing the dilution, and accelerating the bactericidal possibilities of the sea water and that, therefore, the place at which stratification breaks down marks approximately the outer limits of the field. This limit may be some distance beyond that place, however, and the actual value of S at its limits is much greater than Equation (22) would indicate at such location. For practical purposes, however, it is considered sufficient to neglect this thickening and apply Equation (22) on the assumption that S equals from 200 to 225.

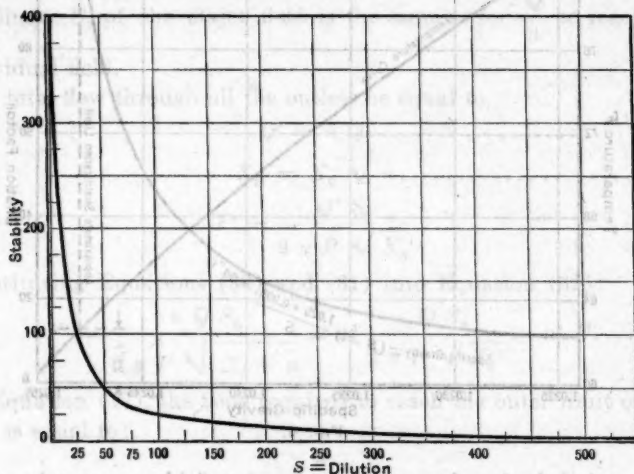


FIG. 5.—RELATION BETWEEN DILUTION AND STABILITY.

Following the assumption that the field stratification breaks down at a value for S of 200 +, care should be taken to provide sufficient depth of water at the onshore field limit to permit of material field thickening; that is, the depth of sea water at the shoreward limit of the field should be greater than the probable value for P .

CORROBORATION AND CORRECTION FROM RESULTS OBTAINED AT EXISTING OUTFALLS

Reference to Fig. 3 indicates that the upper limit of Curve A follows, within reasonable limits, the results obtained by measuring the actual value of S_0 obtained at four existing California outfall sewers.

At Long Beach, the ocean outfall consists of a 36-in. cast-iron pipe extending 850 ft. seaward along the ocean bed and discharging 8.5 ft. below mean sea level through a vertical riser.* The flow of sewage is continuous and during the series of experiments it varied from 6.7 to 10.5 cu. ft. per sec. The point of discharge is located in exceedingly calm water and conditions over

* Replaced in 1923 by 42-in. Catalina Island outfall into 32 ft. of sea water, because of adjacent shore pollution.

this outfall approach more nearly those under which the experiments at San Pedro were performed, than at any other outfall site investigated.

At Santa Monica, the ocean outfall consists of a 24-in. pipe supported by the Santa Monica Pleasure Pier.* At the end of the pier it discharges vertically downward through a 20-in. pipe to a depth of 13.4 ft. below mean water level. During the period of observation, 7.5 cu. ft. per sec. was flowing, resulting in a velocity through the down pipe of 4 ft. per sec. and for the stage of the tide at that time, a value of L of 14.5 ft.

At Santa Barbara, sewage is discharged from a pumping station through a 42-in. reinforced concrete pipe laid in the ocean bed, the direction of discharge from the end of the outfall pipe being nearly horizontal and discharging the sewage 37 ft. below mean water level. The flow is intermittent; with one pump in operation 10.87 cu. ft. per sec. is discharged while with two, the discharge is 15.2 cu. ft. per sec. During the period of observation one pump was able to empty the sump in about 4 min., and the interval between pumping was about 10 min.

At Hyperion, the City of Los Angeles has constructed a 7-ft. reinforced concrete outfall extending 5 300 ft. seaward from the shore line and terminating in a Y, with branches 320 ft. and 108 ft. long, respectively. This Y was constructed at the end of the outfall to divide the flow of sewage into two streams of equal quantity. Each leg terminates in an upward riser so that sewage is discharged vertically upward.

On September 29, 1926, the City of Long Beach, in co-operation with the California State Board of Health, conducted a bacteriological survey to determine the extent of the polluted area surrounding the ocean outfall. The extent of the field was assumed by a limiting count of 10 *B. Coli* per cu. cm. of seawater sample and was found to comprise 507 acres. The maximum sewage flow during the survey amounted to about 10.5 cu. ft. per sec.; or a polluted area of 48.3 acres per cu. ft. per sec. of sewage discharged. In this value of Q the writers differ with the city officials of Long Beach who state that the maximum was 15.3 cu. ft. per sec. Flow was computed by measuring head loss in passing a Republic flow meter disk, the recording apparatus being out of order at the time.

Equations (1) to (35) are not applicable to the determination of the size of a field under conditions existent at Long Beach. They would indicate a field far too large and one in which bacteriological destruction would occur long before the calculated outer limit of the field at $S = 200$ would be reached. Furthermore, they would indicate that a very thin layer of mixed water would have to maintain itself as an independent stratum for a period extending into days. Influenced by the tidal currents into and out of the Long Beach Inner Harbor Entrance, sewage from the Long Beach Outfall would reach the outer limits of the actual field, as found, in 4 to 6 hours.

At the same time that the City of Long Beach was conducting the bacteriological survey the writers made a separate survey of dilutions and surface velocities within a radius of 200 ft. of the outlet. The sewage is discharged

* To be replaced in 1929 by connection to Los Angeles City System because of adjacent shore pollution.

between the two breakwaters which protect the Inner Harbor Entrance and about 200 ft. east of the channel into the Inner Harbor; it is, therefore, subject to an eddy current. Measurements of the dilution were made in three directions from the column head, from which it was found that the value of S was $2.8 x^{0.31}$, $0.58 x^{0.32}$, and $2.5 x^{0.32}$ in the respective lines of travel. The differences were due to the effect of the eddy current. Substitute in Equation (29), the known value of $S_0 = 2$, a mean value of $S = 1.2 x^{0.5}$ and $n = 1$; then, x cancels and $X_0 = 2.8$. By applying the same values in Equation (23), X_0 is found to be equal to 2.6 which agrees with 2.8 within the limits of error allowed for the observation.

A rough check on the thickness of the field was obtained from the velocity at which the flow of sewage left the column head. This was made by observing the time required for chips of wood to float past fixed points in one direction from the outlet. The observations follow the formula, $t = 0.08 x^{1.58}$, which, by differentiation, results in the equation:

$$V = \frac{7.9}{x^{0.58}} \dots \dots \dots (36)$$

The field thickness, P , may be obtained by first combining Equations (19) and (36) and then substituting the known values, $Q = 14.3$ and $S = 1.2 x^{0.50}$. For $x = 15$ ft., the value of P is found to be 0.42 ft. and for $x = 200$ ft., $P = 0.36$ ft. Assuming the mean velocity of the sewage stratum to be 0.8 of the surface velocity, as in float measurements, the field thickness at 15 ft. becomes 0.53 ft. as compared to 0.75 ft. obtained from Equation (16). This observation of field thickness is noted for the purpose of illustrating the shallow depth of the sewage field.

Twice each year the City of Los Angeles conducts a bacteriological survey in the vicinity of its ocean outfall at Hyperion, which is intended to determine the area of ocean pollution. Owing to the fact that shortly after the construction of the outfall numerous leaks of considerable magnitude appeared at the joints between concrete pipe sections and that the leaks were not repaired until about April, 1927, there are only five of the surveys which interpret conditions as they should exist with the sewage discharge confined to definite, known outlets at or near the seaward end.

TABLE 3.

Date.	Q , in cubic feet per second.	Observed area of field, in acres.	Acres per cubic foot per second.	Number of outlets.	Depth, in feet.	S_0 .	Computed area, in acres.
January 11, 1925..	120	720	6.0	1	57	10.5	710
May 17, 1927.....	200	555	2.7	3	56	14	524
October 4, 1927....	170	986	5.8	2	55	11.5	(986)
February 29, 1928..	197	834	4.2	4	52	15	1 090
May 23, 1928.....	205	1 075	5.2	4	52	14.5	1 170

In Table 3 are the results of these five surveys, together with the theoretical results supplied by application of Equations (1) to (35) to conditions as they existed on the dates of the survey.

The comparison of the field spread in each case (see Table 3) is made on the assumption that the value of X_0 is alike in each case, that it is correct within the reasonable limits of error, and that the outer boundary of each field is fixed by a value of $S = 212$. This was the value of S at the field boundary as determined by Equations (1) to (35) for the survey conducted October 4, 1927.

The advantage of initial dilution is well indicated by a comparison of these three fields with each other and with those at Long Beach. Between February or March, 1925, and April, 1927, the line leaked badly and could readily be compared with a line having a large number of small outlets. Each bacteriological survey conducted during this period indicated a smaller area of pollution per second-foot of discharge than is shown by any one of the five surveys given in Table 3.

At Deer Island, in Boston, Mass., Harbor, about 1920* the sewage from a population of 700 000 was discharged horizontally in 30 ft. of water through fourteen rectangular outlets, 18 by 30 in. in size, distributed through a distance of 120 ft. Observations showed that the value of S_0 was 21 and that 300 ft. away from the outfall pipe, sewage could not be visually detected. Several hundred feet away, chemical analysis showed about 100% sea water.

Assuming a rate of 100 gal. per capita daily, from the tributary population, the maximum total flow through the Deer Island Outfall would amount to 108 cu. ft. per sec., or 7.7 cu. ft. per sec., flowing through each outlet. The corresponding length of path should be 36 ft. and, assuming that the outlets are arranged in pairs discharging in opposite directions so that rising columns do not interfere, $S_0 = 19$ theoretically. For $x = 300$, $S = 110$, and for $S = 212$, $x = 2100$ ft. and the field area, 317 acres, or 2.94 acres per sec-ft., corresponding to a field limited by the same conditions of final dilution as the observed field at Hyperion. If the observed value ($S_0 = 21$) of the initial dilution is used, the field limit, where $S = 212$, is found to be at $x = 1715$ ft. and the area is 212 acres. As far as can be determined from data available, Equations (1) to (35) indicate a field on the side of safety.

PROBABLE ERROR

The probable error in the value of S_0 , as determined from several of the vertical observations at constant depths, amounts to approximately 4 per cent. If a second variable is introduced, which in this case is the length of path, it would cause this to be multiplied by $\sqrt{2}$, making the resulting probable error about 6 per cent.

Since x , the distance to the outer boundary of the field, is inversely proportional to the square of the initial dilution, it is subject to an error of 12%, and the area of the field to an error of 25 per cent. Therefore, while so much depends on the value of S_0 , it is a useless refinement to compute its value, and it can be found from Fig. 5 within the limits of error.

All the experiments at San Pedro Harbor were performed in very quiet water, and at Long Beach, the water at the outfall discharge is almost equally

* Report of Special Sewage Disposal Commission, Los Angeles, Calif., August 10, 1921.

as smooth. At Santa Barbara there are small swells and the outfalls at Santa Monica and Hyperion are in the open ocean. It has been considered axiomatic that, other conditions being equal, discharge into rough, disturbed water would result in greater dilution than discharge into calm water. The experiments at San Pedro indicated this clearly, in that relatively larger values of S_0 were obtained with the use of $\frac{1}{4}$ -in. and $\frac{3}{8}$ -in. nozzles, than with the large sizes. The explanation for this is that, even in the more sheltered area, there were always present, currents of sufficient magnitude to effect results from these small streams of discharge, and these had little effect on the larger ones.

Extending this reasoning to nozzles of a size likely to be used in outfall sewer construction, it may be seen that on a calm day little, if any, increase should be found over the theoretical value in the dilution. When pollution is fixed by its effect on neighboring bathing beaches, calm weather conditions should be taken especially as a criterion for obvious reasons.

APPLICATION OF EQUATIONS (1) TO (35)

If the horizontal jets are to be used, the length, L , may be found from Equations (8) and (9) by introducing the value of x given in Equation (7); or, as circumstances warrant, may be taken from Fig. 6. For vertical jets, $L = y$.

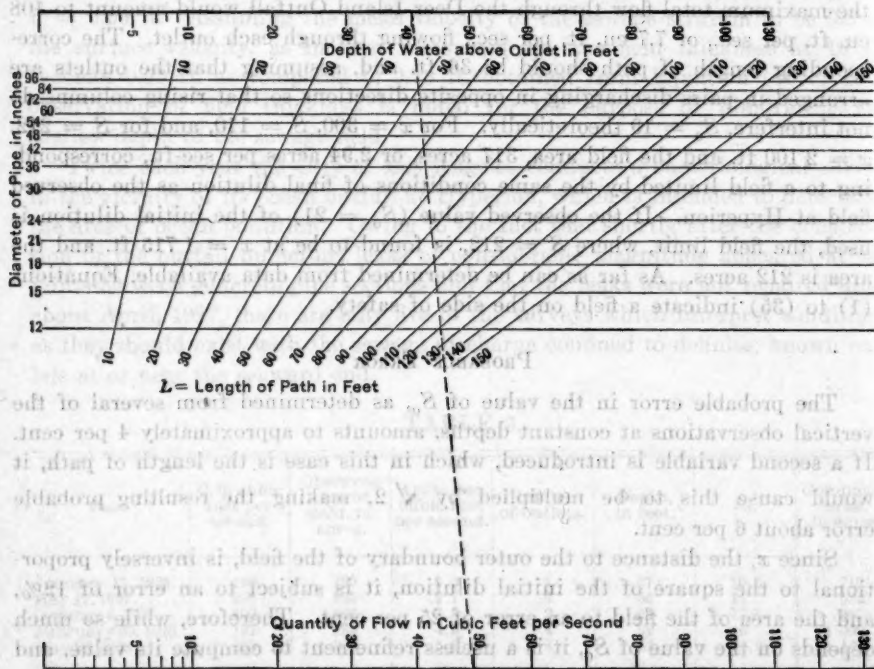


FIG. 6.—LENGTH OF PATH FROM OUTLET TO WATER SURFACE FOR HORIZONTAL OUTLETS.

The initial dilution, S_0 , may be found from Equation (10), or Fig. 7. (It should be noted that in Equation (10), Q is in gallons per minute.) The total

quantity of water coming to the surface is $Q S_0$, of which, $Q (S - 1)$ is salt water.

The distance to which the sewage will travel away from the outlet, if there are no currents present, is found from Equations (23) and (29). This distance may also be taken from Fig. 8 with $S = 225$. For any other value of S multiply x by $\left(\frac{S}{225}\right)^2$. In some cases, x and t may exceed the limits of Fig. 8, but then the time element will be so great that other factors will enter and make the formulas inapplicable.

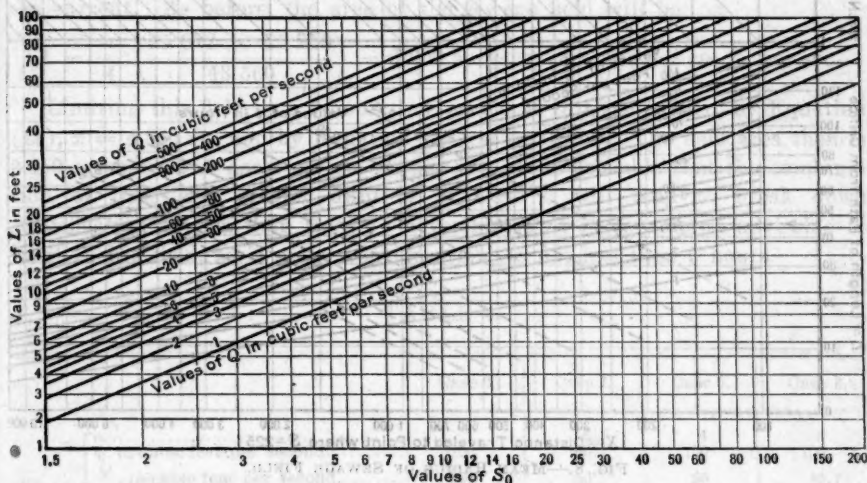


FIG. 7.—INITIAL DILUTION OF SEWAGE IN SALT WATER.

The distance the sewage will travel with the current may be determined from Equation (28), in which, x and t are found from Fig. 8 and U is the velocity of the ocean current.

If it is desired to determine the distance that the sewage will travel against an ocean current, use may be made of Equations (16), (21), and (27).

To compute the time required by the sewage to reach the outer boundary Equation (17) is applicable.

EXAMPLES

As an illustration of the application of these equations assume that 100 cu. ft. per sec. is to be discharged at a depth of 40 ft. below the ocean surface, at a location where a directly onshore current of 30 ft. per min. is considered.

Case 1.—First, consider that the discharge is vertical, through a single outlet, 72 in. in diameter; then, $Q = 100$; $L = y = 40$ ft.; $D = 72$; $n = 1$; $U = 1800$ ft. per hour, or 30 ft. per min.; and, $S = 225$.

From Fig. 8 the radius of the field at $S = 225$ is more than 10 000 ft., so it will be necessary to compute x and t . Ordinarily, this is not worth while as other factors will tend to limit the field, but the computation is carried out as an illustration.

From Fig. 7, $S_0 = 5.8$. By Equations (23), (29), (16), (21), and (17): $X_0 = 8.0$; $x = 12\ 000$ ft.; $p = 3.3$ ft.; $F = 9.9$; and $t = 24.5$ hours, respectively. Hence, $Ut = 44\ 500$ ft., and, from Equation (28), $e = 56\ 500$ ft., which is the theoretical distance.

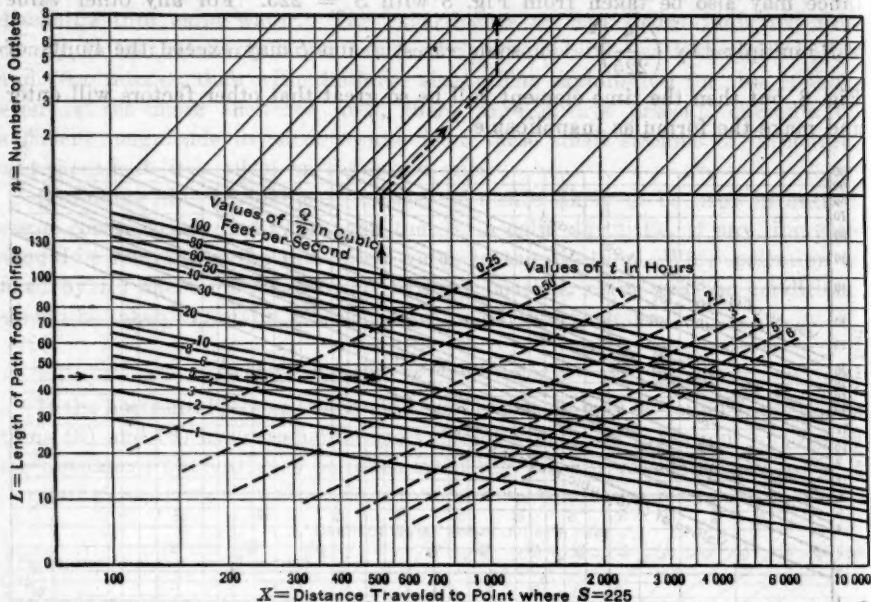


FIG. 8.—MEAN RADIUS OF SEWAGE FIELD.

By Equation (27), $e = 131$ ft., which is the distance the sewage will flow against the current. The shape of the sewage field will be roughly an ellipse with the major axis equal to the sum of the distances with and against the current, or $56\ 630$ ft., and the minor axis, $2x = 24\ 000$ ft. The area will be, $A = \frac{\pi}{4} \left(\frac{56\ 630 \times 24\ 000}{43\ 560} \right) = 24\ 500$ acres, or 245 acres per cu. ft. per sec.

It is very probable in the case under consideration, that the thin stratum will not exist as such for long and that time will limit the field. Assuming 5 hours as the field limit, then, from Equations (17) and (28), respectively, $x = 4\ 150$ ft. and $e = 13\ 150$ ft. From this value the area is found equal to 1 990 acres, or 19.9 acres per cu. ft. per sec.

Case 2.—Assuming the use of two vertical 54-in. discharge nozzles, let $Q = 100$ cu. ft. per sec., $n = 2$; $\frac{Q}{n} = 50$ cu. ft. per sec.; $y = L = 40$ ft.; $D = 54$ in.; $U = 1\ 800$ ft. per hour, or 30 ft. per min.; and $S = 225$.

To prevent interference between the rising columns below the surface, the two outlets should be at least $\frac{40}{3} + 4.5 = 17.8$ ft. apart. The value of S_0 , in this case becomes 8.4. This factor is not needed in the computation, but is

mentioned to emphasize the effect of S_0 on the size of the sewage field. By Fig. 8, $x = 7600$ ft. The time being in excess of 6 hours, is not shown in Fig. 8 because the size of the field will probably be limited by other factors.

By means of Equations (23), (16), (21), and (17): $X_0 = 7.25$ ft., and $X_0 \sqrt{n} = 10.25$ ft.; $p = 3.3$ ft.; $F = 12.6$; and $t = 9.55$ hours, respectively. Hence, $Ut = 17200$ ft.

By Equation (28), $e = 24800$ ft., the distance sewage will travel with the current and by Equation (27) $e = 212$, the distance it will travel against the current. As before, the area of the sewage field will be,

$$A = \frac{\pi}{4} \left(\frac{25012 \times 15200}{43560} \right) = 6900 \text{ acres, or 69 acres per cu. ft. per sec.}$$

Limiting this field by a 5-hour time period as in Case 1, then, by Equation (17), $x = 4850$ ft., and by Equation (28), $e = 13850$. The field area then is 2450 acres, or 24.5 acres per cu. ft. per sec. It is probable that the combination of time and greater dilution in Case 2 will tend to break down stratification earlier than Case 1; nevertheless, it is considered advisable to limit such a field, as is under consideration, to not less than 5 hours.

TABLE 4.

	Case 3.	Case 4.	Case 5.	Case 6.
Given: $\left\{ \begin{array}{l} n, \text{ in cubic feet per second} \\ Q, \text{ in cubic feet per second} \\ y, \text{ in feet} \\ D, \text{ in inches} \\ U, \text{ in feet per hour} \end{array} \right.$	$\left\{ \begin{array}{l} 1 \\ 100 \\ 100 \\ 40 \\ 72 \\ 1800 \end{array} \right.$	$\left\{ \begin{array}{l} 2 \\ 100 \\ 50 \\ 40 \\ 54 \\ 1800 \end{array} \right.$	$\left\{ \begin{array}{l} 4 \\ 100 \\ 25 \\ 40 \\ 30 \\ 1800 \end{array} \right.$	$\left\{ \begin{array}{l} 6 \\ 100 \\ 16.7 \\ 40 \\ 27 \\ 1800 \end{array} \right.$
By Fig. 6: L , in feet.....	58	55	57	55
By Fig. 7: S_0 , in feet.....	11.5	15.5	26	30
By Fig. 8: x , in feet.....	3600	2700	1860	1100
By Fig. 8: t , in hours.....	3.25	1.75	0.47	0.33
Ut , in feet.....	5850	3150	840	600
$e = x + Ut$, in feet.....	9450	5850	2200	1700
X , in feet.....	10.25	9.1	8.38	8.1
$X \sqrt{n}$, in feet.....	10.25	12.9	16.75	19.8
P , in feet.....	4.8	4.6	4.75	4.6
F , in feet.....	11.9	15.45	31.3	23.3
$\frac{1}{3} \left(\frac{F}{U} \right)^2$, in feet.....	203	320	605	725
A , in acres.....	1250	601	137	88
$\frac{A}{Q}$, in acres.....	12.50	6	1.4	0.9

If horizontal outlets are used, the computation is the same as for vertical outlets, except that it is first necessary to determine the length, L , of the rising column from Fig. 6, or by means of Equations (8) or (9). Equation (9) applies only to shallow depths with high velocities at the outlet, where the horizontal distance from the outlet to the point where the column reaches the surface is equal to or greater than three times the depth. For most cases Fig. 6 will suffice instead of Equation (8). Four other cases are given in Table 4.

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A NATIONAL RECLAMATION POLICY:

EXPLANATORY STATEMENT REGARDING REPORT OF THE COMMITTEE OF THE IRRIGATION DIVISION*

By J. B. LIPPINCOTT,† M. Am. Soc. C. E.

In the preparation of its report on a National Reclamation Policy, the Committee of the Irrigation Division considered many points which it decided should not be elaborated or even mentioned in the report itself. The following explanatory statement is intended to give the report its proper setting. These remarks, therefore, may be considered as parenthetical or supplementary to the official report.

The Reclamation Act was passed on June 17, 1902, for the purpose of utilizing funds derived from the disposal of public lands, for the irrigation of arid lands in sixteen Western States. On June 12, 1906, the Act was extended to Texas. The law provides (Section 4), that,

"* * * upon the determination by the Secretary of the Interior that any irrigation project is practicable, he may cause to be let contracts for the construction of the same. * * * The said charges [not exceeding 10 annual payments] shall be determined with a view of returning to the reclamation fund the estimated cost of construction of the project [without interest]."

The Act has been extensively amended. The provision for non-payment of interest has been retained, but the time of payments of construction charges was extended first to twenty, and then to forty, years. Extensive waivers of construction charges also have been made. In 1914, when Secretary Lane secured the Reclamation Extension Act, the law was changed and not only the adoption of new projects, but all appropriations therefor, had to be passed by Congress. Secretary Work on several occasions declined to construct projects so authorized, which in his judgment were not economically feasible.

* Presented at the meeting of the Irrigation Division, San Diego, Calif., October 4, 1928. Written discussion on this paper will be closed in September, 1929.

† Cons. Hydr. Engr., Los Angeles, Calif.

In order to obtain a general perspective of the problem involved the following data are presented:*

Total area of crop land in the United States	
in 1924	391 647 372 acres
Crop land idle (8.7%).....	34 007 716 "
On the Pacific Coast in 1924 the total area of	
crop land was.....	18 715 791 "
The crop land idle was 28% of this, or.....	5 201 519 "

According to the report of the United States Reclamation Service for 1926 the total area provided with water under Federal Reclamation Projects was 1 803 000 acres, of which 483 000 acres, or 27%, was idle. It has been stated† as follows:

Irrigable land not now irrigated (in the United	
States	30 000 000 acres
Wet land requiring drainage only.....	30 000 000 "
Wet land requiring drainage and clearing.....	60 000 000 "
Total	120 000 000 acres

From this it may be seen that the reclamation problem remaining is a large one.

The report of the Special Advisory (Fact Finding) Committee to the Secretary states‡ that up to June 30, 1923, the original construction cost of projects was \$147 831 506. It further states: "The facts developed show that \$18 861 146 will never be recovered. There will be a probable loss of an additional \$8 830 000." The works built have been a credit to the Engineering Profession, the fund has been honestly administered, and the economic benefits to the West have been great. Such an extended experience is a justification for a review of the methods and results and for a discussion of the lessons that may be drawn therefrom.

WAIVING OF INTEREST PAYMENTS UNWISE

As shown by the debates in Congress at the time of the adoption of the Act, and by the policies subsequently approved, Government irrigation enterprises were confined largely to projects which at that time were too expensive and required too long to construct to make them feasible as private enterprises. They then became feasible only by a system of investment that neglected the item of interest.

The extent of the subsidy due to this waiving of interest that practically has been granted to the water users on the Federal projects, as compared to

* Year Book, U. S. Dept. of Agriculture, 1925, p. 1380.

† Loc. cit., 1921, p. 430.

‡ Senate Doc. No. 92, 68th Cong., 1st Session, p. 53.

the costs borne by settlers in State districts, or on private projects, is indicated by the following cost of United States Reclamation Work to June 30, 1922:*

Reported debits.....	\$171 496 409
Reported credits.....	41 350 449
Reported net investment.....	\$130 145 960
Corrected net investment (4% interest on annual net investments, compounded, to June 30, 1922)...	70 706 685
Total cost.....	\$200 852 645

The annual interest on the reported net investments plus interest on previously accumulated interest for 1922, according to the same authority, was about \$7 725 000, or nearly twice the anticipated annual repayments on construction charges. Due consideration, however, should be given to the great indirect benefits that have been created by these works.

A computation of the amounts which would be due if interest at 6% were charged on deferred payments, shows that a farmer not so benefited would pay between 72 and 66% more for his irrigation works than one operating under the Federal Reclamation Law.† This does not provide for compound interest which would further increase the amount. Neither does it take into consideration the time during which water was supplied to farmers on Federal reclamation projects before payment of construction charges began, which in some cases has reached fourteen years.

According to the Fourteenth U. S. Census, 80% of the land irrigated in 1919 in arid parts of the United States was supplied with water by private enterprises that had received no public aid or endorsement; 6.5% had been developed by the use of Federal funds; 9.5% was served by State irrigation districts; and 2.7% was under Carey Act grants.

There is marked special legislation in selecting the settlers on 6.5% of the total irrigated area for the granting of such subsidies, while the settlers in the remaining 93.5% of the enterprises have paid full costs. If other irrigation enterprises were prosperous, such an aid to the settlers on Federal projects would not be so conspicuous, but it becomes so, because many irrigation districts and private enterprises are struggling for existence.

It appears unfair that the farmers on 392 000 000 acres of land in the United States should be taxed for the purpose of supplying a large subsidy to those on only 1 803 000 acres, supplied with water by the United States. This inequality is emphasized as there is an over-production in practically every branch of agriculture. It is the surplus, even although small, which breaks the price. The granting of subsidies to individuals also tends to destroy their ambition and to make them still further dependent. The report of the Special Advisory Committee (Fact Finding) in 1924, truly states (page XIII):

"The Reclamation Service has retained the full management of all but two of the projects. This has not been satisfactory. The project management and the Washington office have become targets for criticism. A dependence

* Bulletin No. 1257, U. S. Dept. of Agriculture, p. 8, et seq.

† Bulletin No. 1257, U. S. Dept. of Agriculture.

on Federal paternalism has settled down upon nearly all the projects, and a corresponding bureaucratic tendency has grown up within the Reclamation Service. The water users have come to look upon themselves as wards of the Government, a specially favored class with special claims upon governmental bounty; and the Reclamation Service has been tempted to accept this definition of the water users. Nothing could be more detrimental to the progress of a venture which demands, first of all, individual courage and independence of the people concerned. The extension act provides that the operation and maintenance of the project may be turned over to the water users. This should be done at the earliest possible date."

The Bureau now recognizes that the management of projects should be promptly transferred to responsible local agencies; and this is being done.

Notwithstanding the waiving of this interest charge, large numbers of the settlers on the Federal reclamation projects have not paid their construction charges on projects costing on the average \$57.75 per acre.* In addition, the "Omnibus Adjustment Act" intended to adjust water right charges, passed on May 25, 1926, made large deductions for construction charges on twenty Federal projects. Such conditions would not encourage the ordinary banker in making further loans to settlers as has been advocated in other parts of the Fact Finders report. Neither does it offer an attractive prospect for the repayment for projects now adopted which it is said will cost from \$150 to \$200 per acre.

It was the unanimous opinion of the Committee on a National Irrigation Policy that farmers on reclamation projects should pay interest charges on the cost of their works as do the water users on the remaining 93.5% of the irrigated areas of arid America. It is not fair to give the new settler preferential privileges above the water users on the State irrigation districts or under private enterprises. The rate of interest charged could properly be that at which money costs the Government itself, but the Government should not be called upon to sustain this loss in order to benefit individuals. The experiments tried in California and in the State of Washington do not indicate that paternalistic treatment of settlers has resulted in any particular benefit to them.

Under the California Irrigation District Act the bond issues which have been delinquent in the payment of interest and principal in 1927, when this subject was investigated by the writer, amounted to \$4 627 000, on 95 810 acres. Their total bond issues amounted to \$137 000 000, on 3 915 572 acres. The delinquency therefrom was but 3.4 per cent. The prevailing bond interest rate is 6 per cent. The California law provides the same general method for the sale of lands on which payments are delinquent as with other taxes.

The report of the Special Advisory Committee (Fact Finding) relative to Federal irrigation projects states (page XI):

"On June 30, 1923, of the construction charges then due, 14.2%, or \$2 537 222.46, remained unpaid, and of the operation and maintenance charges then due 17.6%, or \$2 423 649.06, remained unpaid."

Of late, the Bureau has taken a stronger stand to enforce the payment of these charges.

* Senate Doc. No. 92, 68th Cong., 1st Session, p. 146.

In the case of California irrigation districts where the law is administered locally, payment of taxes is insisted upon and where there are no subsidies, better financial results have been obtained. However, in the arid area of the United States as a whole, 26% of all irrigation districts organized have failed in some period of their development, so that this method of procedure has also had its disasters. At first, these district laws were loosely drawn, and there was almost no supervision of their procedure in voting and selling bonds or letting contracts. In California, there are now close supervision and few failures. The Committee is of the unanimous opinion that a subsidy to settlers is not advisable. Where the Government has made commitments they should be honored, but the same fulfillment of contracts should be required of the water user.

The most flagrant examples of repudiation on Federal projects are said to be cases where local influences from the start undertook to secure public expenditures for reclamation with no intention of making repayments as required by their contracts. Some depended upon legal flaws, but others mainly relied on political influences. This movement made no progress in Congress as long as the Interior Department stood firm for the enforcement of the law and the contracts. These waivers are said to have been first covertly and then openly advocated in past years by the Department. When this succeeded, it of course broke the phalanx of resistance and many scrambled for such benefits. When the morale was broken the enforcement of contracts became more difficult.

It is a desirable and necessary public policy for the Federal Government to build hydraulic works whether for irrigation, drainage, or navigation, for the general benefit of the greatest possible number of citizens, rather than to make these allotments of funds for individual settlers on selected areas. This matter will be discussed further.

THE COLLECTION OF PHYSICAL DATA

Surveys of reservoir sites can be made promptly. Dam sites can be explored and distribution systems can be outlined quickly. However, the data necessary for a proper outline of a project, such as stream flow, rainfall, and temperature records, can only be obtained through years of continuous observation. Private enterprises are not adapted to the collection of such data; it is the province of the State and Federal Governments to make such long-time observations. The stream-gauging work of the U. S. Geological Survey is especially to be commended. It should be expanded. Some of the Western States are properly co-operating with the Federal Departments in collecting such physical data and in making essential topographic surveys.

REGULATION OF STREAMS SHOULD BE UNDER PUBLIC AUTHORITY

The determination by the Federal Government of the best comprehensive plan for the utilization of drainage systems where streams are interstate or navigable, is very important and such work is commended. It has adopted a broad policy of improving its navigable waters. Great expenditures have been made for this work. This is justified because of general public interest. Of late, many localities benefited have contributed to the cost of such improve-

ments. The fact that certain individuals are especially benefited is not taken as the occasion for their direct assessment. A similar policy has been adopted for the improvement of main highways and large appropriations have been made therefor. Here, again, there is a co-operation between the Federal and local authorities. The Bingham Act provides for the aid of aviation in a manner similar to the Federal aid of navigation. As an example, the Federal Government now has a lighted air lane extending from New York, N. Y., to San Francisco and Los Angeles, Calif., and also one nearly completed from Los Angeles to Seattle, Wash. These air lanes are lighted at night and are supplied throughout their courses with emergency landing fields at Federal expense. The large cities served are requested to furnish the terminal airports.

Congress at its 1927-28 Session passed* "An Act for the Control of Floods on the Mississippi River and Its Tributaries and for Other Purposes". An appropriation of \$325 000 000 was authorized. The Act provides that "no local contribution to the project herein adopted is required." This generosity, it is stated, is due to the extent of the problem involved, and because of large previous local expenditures. Section 4 provides that benefited localities are required to supply only certain minor rights of way. The aid of navigation is not the purpose of this Act, which contemplates the construction of regulating reservoirs. Section 3 provides for the States or levee districts to maintain the flood-control works after their completion and defines maintenance as including normally such matters as cutting grass, removal of weeds, local drainage, and minor repairs of main river levees.

These provisions are in distinct contrast to the proposed Act for building the Boulder Canyon Reservoir on the Colorado River where the full return of all funds to the Government is required by firm contracts in advance.† Section 13 of this Act also provides an appropriation of \$17 600 000 for the control of the floods in the Sacramento River for which no return is to be made. This latter work is part of a co-operative act between the State of California, local land owners, and the Federal Government. Other great projects are proposed for the improvement of inland waterways at Federal expense and without return of funds.

The Federal policy of making large direct appropriations for public works for the general development of broad sections of the country, for navigation, flood control, highways, and aviation, has been established without repayment of the amounts furnished by it. Some such Federal policy should be adopted for the general benefit of all interests, including irrigation on navigable or interstate streams or their tributaries as well. Storage reservoirs and river-control works could properly be built in large part by Federal and State funds on such rivers as the Columbia, Sacramento, Colorado, Rio Grande, and their tributaries; and the floods could be regulated for all public use (be it irrigation, navigation, water supply, or power) and in such manner that the greatest possible numbers may be aided.

The Federal Government has already expended large sums for river-control works on the Lower Colorado by direct appropriations. Navigation in this instance is but a myth. If it becomes necessary to build an All-American

* Public Doc. 391, 70th Cong., 1st Session.

† H. R. 5773, 70th Cong., 1st Session, 4-b.

Canal because of international complications to protect the irrigators of the Imperial Valley and to save just portions of the waters of that stream for the development of public and private lands in the United States, it appears fair that a direct appropriation should be made for that purpose. The people have done their full share in this effort, as well as in the projects of the Mississippi Valley. Established institutions are fully as deserving of help as new projects. This would be a broader and franker policy than the selection of 6.5% of the irrigated lands for special subsidies. The general plan for such works on interstate and navigable streams should be adopted jointly by the Federal and State Governments.

On non-navigable streams wholly within the State such works could properly be aided by the State itself, or by large conservation districts, or by the two combined. Some part of the cost of such works should be allocated to the interests benefited, but a substantial part of the expense for river regulation should be at public expense.

CONSTRUCTION OF WORKS BY PUBLIC OR PRIVATE AGENCIES

Municipalities or other public agencies, or private parties, should be permitted and encouraged in building hydraulic works; but the control of large reservoirs built by Federal authority should remain permanently under its control because of the varied interests involved in their operation. This is the present provision of the Reclamation Act. When built as a part of a comprehensive plan, in so far as possible, they should be administered so as to produce the greatest general benefit. In the case of the Colorado River project the order of importance adopted is: First, river regulation; second, water supply; and third, power.* When reservoirs are built by private funds it is proper to provide for their ultimate recovery by the public as provided in the Federal Water Power Act.

WATER AND POWER FROM FEDERAL OR STATE RESERVOIR SHOULD BE DISPOSED OF AT WHOLESALE RATHER THAN RETAIL

All the members of the Engineering Committee were agreed that State and Federal authority should not properly engage in the distribution of water or power to the individual consumers. The writer sees no reason why governmental authority should be restrained from installing the generating plants. It is not intended that water and power from Federal and State projects should be turned over only for distribution to private parties, but rather that it should be also delivered to either political subdivisions of the State or to mutual or private companies which have direct contact with the consumer. In cases, such as in the Salt River Valley Project, where an existing privately owned power distribution system was already in operation, it became naturally a medium for the distribution of power, just as the Salt River Valley Water Users Association with its network of canals became the natural distributor of water.

* H. R. 5773, 70th Cong., 1st Session.

The Congressional Act of May 25, 1926, Section 46, provides:

"No water shall be delivered upon the completion of any new project or new division of a project until a contract or contracts in form approved by the Secretary of the Interior shall have been made with an irrigation district or irrigation districts organized under State law providing for payment by the district or districts of the cost of constructing, operating, and maintaining the works during the time they are in control of the United States."

Section 45 of the same Act provides for a similar procedure in the case of amendments to any existing water-right contracts. While the water is to be delivered wholesale by the Reclamation Service to the district, its actual distribution to the consumers and the collection of bills are very properly to be undertaken by a local organization and not by the Federal Government. This policy is to be commended. Section 5 of the Act entitled, "H. R. 5773, 70th Congress, 1st Session", relative to the building of the Boulder Canyon Dam provides as follows:

"That the Secretary of the Interior is hereby authorized, under such general regulations as he may prescribe, to contract for the storage of water in said reservoir as may be agreed upon, for irrigation and domestic uses, and generation of electrical energy and for the delivery thereof at such points on the river and on said canal and delivery at the switchboard to States, municipal corporations, political subdivisions, and private corporations, of electrical energy generated at said dam, upon charges that will provide revenue which, in addition to other revenue accruing under the reclamation law and under this Act, will in his judgment cover all expenses of operation and maintenance incurred by the United States on account of works constructed under this Act." (Section 4-b) "* * * and the repayment, within 50 years from the date of the completion of said works, of all amounts advanced to the fund * * * with interest thereon."

As far as the policy of wholesaling the water or power as provided in this bill is concerned, it is in accordance with the judgment of the Engineering Committee. However, the provisions of this Boulder Canyon Act are in marked contrast with the enormous appropriations for the Mississippi River where the obligation of the beneficiaries is to cut the grass on the levees.

BENEFITS FROM PUBLIC FUNDS SHOULD BE EXTENDED TO ALL WATER USERS ON THE STREAM

It has been stated* that 34 007 716 acres of the "crop land" of the United States were lying "idle or fallow". In the "Mountain States", which included Montana, Idaho, Wyoming, Colorado, New Mexico, Arizona, Utah, and Nevada, out of 25 604 416 acres of "crop land", 3 705 498 acres were "idle or fallow". The area of "land in farms" increased between 1920 and 1925, in the Mountain States, 12.4% and, in the Pacific States, it decreased 3.3 per cent. The average for the United States for this period showed a decrease of 3.2 per cent.† In 1920 there was a gain of 500 000 in farm population in the United States; in 1921, this gain was 200 000; in 1922, the loss was 460 000; in 1923, 461 000; and in 1924, it was 182 000.‡ The decrease in farm population in 1924 due to the city-ward movement was highest in the Mountain States, or 4.3%, followed next by the Pacific and West Coast Central States.‡

* Year Book, U. S. Dept. of Agriculture, 1925, p. 1380.

† Loc. cit., p. 1379.

‡ Loc. cit., p. 33.

Between 1910 and 1920 the increase in the total population of the United States was 13 738 354, or 14.9 per cent. A table has also been published* giving acreages, production, and farm values from 1924 to 1925 which shows an estimated decrease in farm values of \$570 663 000. From these data it appears that agriculture is unprofitable and farming consequently unpopular.

In view of these conditions, Secretary Jardine stated:†

"I am opposed to bringing new areas under cultivation until we have found a market for the products we are now producing. There are two things the Government may do to prevent injudicious expansion of farm land. It may regulate its own land-settlement projects wisely and it may discourage undesirable private projects. There is a field here for useful and legitimate Government activity for the protection of agriculture and the promotion of the general welfare. I am as strongly in favor of such activity as I am against attempts to determine economic law by means of legislation."

In operating United States Reclamation projects, 29% or more of the land under canal is not farmed.‡ In California State irrigation districts, one-third of the land is uncultivated.

As water becomes more valuable, the methods of its use are improved. A substantial conservation of water in the future may be accomplished by the lining of canals. It is also being found that larger crops can be grown by the use of less water. Because of these conditions it is probable that extensions of existing canal systems will be made without increasing the volumes of water now being diverted. A notable example of such improved efficiency is in the Salt River Valley, in Arizona.

Due to the work of the U. S. Department of Agriculture there is an increase in the quantity production per acre in agricultural lands because of improved seeds. There has also been a great advance in efficiency of farm equipment which permits of deeper plowing and more effective harvesting. This progress is now well under way and will help, if it does not completely meet (from existing farm areas), the increasing demands for foodstuffs. This further relieves the immediate necessity for increasing reclaimed areas.

Throughout the United States, farm and orchard products have been low priced since 1923. While the prices on the staples, such as corn, wheat, and cotton, are determined by world-wide conditions, the crops that are best produced on higher priced lands, consisting largely of fruits and vegetables, are fixed in the home markets. A small excess of the supply above the demand breaks the price and conversely a slight deficiency stimulates it. During this period there has been an excess of the supply over the demand, especially with fruits and vegetables. This has resulted in losses and farm abandonment. This condition is one which should not be aggravated by the reclamation of new areas.

Nils A. Olsen, Assistant Chief of the Bureau of Agricultural Economics, U. S. Department of Agriculture, has stated§ in part:

* Year Book, U. S. Dept. of Agriculture, 1925, p. 1356.

† Loc. cit., p. 22.

‡ Senate Doc. 92, 68th Cong., 1st Session, p. 26 (Fact Finders Report.)

§ Proceedings, First Pan-Pacific Conference on Education, Rehabilitation, Reclamation, and Recreation, Honolulu, Hawaii, April, 1927.

"A prosperous agriculture can not be achieved by over-development and over-expansion which injures not only established farmers but also new settlers. * * * The process of pushing agricultural expansion too rapidly tends to defeat its own end, if that end is the establishment of prosperous families on the land. * * * The idea of promoting residence in the country unless that residence can be maintained on a profitable basis to the family is to expect an impossibility and fly in the face of modern tendencies. * * * It is obvious that from many points of view it is much more economic to meet any demand for increased agricultural production by utilizing lands already in farms than by attempting to establish new farms."

It would be a much broader policy for the Federal and State Governments to aid the farmer who is now struggling with adversity on existing farms than to adopt a policy of reclaiming new areas whether by drainage or irrigation. There is a great need to reduce the difference between the price received by the producer and that paid by the consumer.

The argument in favor of the adoption of some of the major new projects which would result ultimately in increasing agricultural production has been stated* by Herbert Hoover, Hon. M. Am. Soc. C. E., in referring to the Lower Colorado River development in the hearings before the House Committee on Irrigation, as follows:

"MR. HAYDEN.—The feeling in Congress is not very strongly in favor of reclaiming vast areas of land. It means in this instance a million acres.

"SECRETARY HOOVER.—I think the answer to that would be that it would require approximately 10 years to complete the engineering work, and another 10 years before a complete use was made of the available water to those valleys, and, therefore, before production becomes very substantial there would be a period of 15 to 20 years. In that period the population of the United States will have increased twenty or thirty million, and that production will be very badly needed. I would be the last to wave the magic wand which would bring the production on at the present moment. It will probably take 20 years to complete."

It will be noted that Secretary Hoover agreed that present conditions do not justify immediate reclamation of additional areas, but he estimates that in 15 to 20 years their products will be required.

The policy adopted relative to the Mississippi River of protecting the farmers of the South who are now struggling with adversity from floods, is humane and proper. In a stream, such as the Sacramento, a dam could properly be built at Kennett, Calif., largely at Federal and State expense, which would sustain navigation, prevent floods, provide an adequate water supply for the existing farms in the Sacramento Valley and lower portions of the San Joaquin Valley, and permit of the generation of enough power to pay at wholesale rates for the greater part of the cost of the project. The distribution of both the water and power could be left to local agencies. Such a procedure would yield a State-wide and nation-wide benefit to as great an extent as the building of harbors or highways.

The Sacramento Valley is now staggering with debts of State drainage projects that probably cannot be paid. The limit of the Valley's irrigation water supply during the summer months has been reached, and navigation has

* H. R. 6251 and 9826, 69th Cong., 1st Session, p. 49.

been rendered impossible above Sacramento. The salt water from San Francisco Bay is encroaching on the reclaimed islands of the lower river. A comprehensive scheme involving the expenditure of large public funds for the betterment of such situations, in which the people of the entire West are involved, is commensurate with State and National effort.

Southern California now has a population of more than 2 000 000. It is increasing at a very rapid rate. Despite the fact that nearly all the surface flow of the streams in this region is used, about 80% of the drinking water has to be obtained from wells in Southern California proper. The groundwater supplies in most localities are falling. The only comprehensive source remaining is the Colorado River which is discharging in greater part in destructive floods into the sea about 16 000 000 acre-ft. per year, threatening the very existence of an irrigation district of 600 000 acres, followed by periods of inadequate low flow. By its control at the proposed Boulder Canyon Reservoir these adverse problems can be solved and made largely to pay for the cost of the river's improvement through the development of power, resulting in the great increase in National, as well as local, wealth. If this is not done there may be decadence instead of growth. The cost will be enormous; including the aqueduct, it may exceed \$200 000 000. Surely such a project is worthy of both State and Federal aid. The financial plan proposed is fully justified, if not too liberal on the part of the local interests benefited.

The cost of projects in the Northwest that have been adopted by Congress during the past few years, involving a construction outlay estimated at approximately \$100 000 000 for the next ten years, are said to require an outlay of \$160 per acre, or more. The water users thereunder are to be given forty years in which to refund this amount without interest. In view of the past history of the Reclamation Service, with projects costing less than one-half this amount, it is the prevailing belief that these repayments never will be made. The sum of \$160 per acre is sufficient to acquire good farming lands in Illinois, Iowa, Kansas, and Nebraska, where schools, roads, and other public improvements already exist and where the markets for the products are at hand and large areas are idle. Under these conditions such projects should not now be built, particularly under the present condition of agriculture. Farmers are not asking for their construction. It appears to be a better policy to aid those who are endeavoring to develop areas now occupied and in need of help.

NEW PROJECTS SHOULD ONLY BE ADOPTED AFTER APPROVAL BY COMPETENT AUTHORITY

The existing law (Reclamation Act of 1902, Paragraph 4) states that the adoption of projects is dependent on the determination by the Secretary of the Interior that they are practicable. The Reclamation Service in the past, as well as at present, has in its organization many able engineers. The Secretaries have been competently advised by these men, whose judgment usually has been followed. Frequently, pressure has been brought upon Congressional representatives to force the adoption of projects. Congress, however, has now undertaken the adoption of projects. The Reclamation Extension

Act (August 13, 1914, Section 16) now provides that expenditures for projects shall not be made except from annual appropriations of Congress. The Secretary of the Interior, in certain instances, because of adverse reports and his responsibility under the original act, has declined to construct projects authorized by Congress, which is a courageous and commendable attitude.

Obviously, no project should be adopted for construction unless it is physically and economically sound. The employment of competent advice both of an engineering and economic nature and its acceptance require no argument as to its propriety.

INTEREST SHOULD BE PAID

In some Western States paternalistic measures under State supervision have been adopted for the establishment of settlers on the land, including the building of houses, barns, and the installation of machinery, the leveling and fencing of the lands, and the guaranty of power bills for pumping charges. The States have not been sufficiently prosperous to cause an expansion of this policy. While success has attended some, they do not appear to have secured any unusual prosperity. Where success has marked such enterprises, the conditions surrounding their initiation and subsequent progress appear favorable enough to have induced it without such paternalistic assistance.

The conclusion to be drawn from these experiences is that a liberal measure of paternalism will not insure success. Such aid will work injury in the long run to the social fabric that will far outweigh any temporary successes that may be credited to such a policy.

A National Reclamation Policy should be built around two fundamentals: First, the creation of equal opportunities; and, second, the stimulation of individual effort.

Any policy that supplies a prop on which the improvident may lean, and that fails to provide avenues whereby the energetic and thrifty can outdistance the less fortunately endowed is foredoomed to failure. If such aid is to be given it should not be given by the Federal Government.

REPAYMENTS SHOULD BEGIN PROMPTLY

In the past, long delays have occurred after the deliveries of water before repayments for construction costs began. Based on the reports of the Reclamation Service, on four of the projects payments began as soon as water was delivered; on eight, the next year following. However, there are two projects in which in 1922, eleven years after delivery of water, payments had not begun and in one, the intervening period was fourteen years. A 10-year period with 6% compound interest represents 77% of the initial amount. At present, the Reclamation Service is insisting on prompt payments and is to be commended for so doing. Apparently, there is no good reason outside of political influence why these delays have been so protracted in the past.

The Fact Finding Commission properly recommends that as soon as water is provided for two-thirds of an area, the owners thereof should be obliged to begin making the payments. No long interval of time should elapse between

the furnishing of water and the declaration by the Secretary that payments are due.

Any violation of this policy will deprive other settlers of equal opportunities and delay the operation of the revolving fund contemplated in the law. These delays put a premium on improvidency and are unfair to those who are able and willing to meet their obligations. The terms of repayment should be adjusted to the reasonable ability to pay. The Department of the Interior has adopted* the policy of decentralization and has executed contracts with thirty irrigation districts and water users associations on sixteen projects for the transfer to local control of the care and operation of the irrigation works of these projects. This, he states, has resulted in a better morale and financial standing on the projects.

LAND SETTLEMENT ESSENTIAL TO SUCCESS

On most new irrigation projects, whether Federal, State, or private, colonization is a serious problem. It is difficult to induce people to settle these new farm lands because generally, under present conditions, agriculture does not pay. If farming were profitable the colonization problem would largely disappear. This same argument applies to the difficulty in financing the farmer. Farming will not pay until the demand for foodstuffs catches up to the supply.

The bane of many irrigation projects, whether publicly or privately conducted, has been land speculation. If the dirt farmer can be given access to the land at its base price before inflation, his chances of success are better than if he has to pay a speculative price for it.

On the Pacific Coast, real estate agencies, with "high pressure" salesmen, frequently charge from 25 to 35% of the sales price of the lands for their commission, and require that their fee shall be paid from the first installment of the purchaser, the real land owner usually having to wait for his compensation from deferred payments. This is hard on both original land owner and the settler. It is very difficult to overcome such speculations.

The policy now being enforced on new projects may offer a greater stimulus to the proper settlement, than any colonization plan previously attempted. Drastic measures are taken to discourage land speculation prior to the adoption of the project, by fixing sales prices of lands therein. A vigorous policy in this regard will produce a wholesome effect. The Government or State can properly insist as one of the conditions of its undertaking a project that every surplus parcel of land within the proposed district be listed with an authorized real estate agency, at a stipulated price that contains no unearned increment. The Government or State, in these recent instances, properly reserves the right to fix the price of such land. These tracts are to be offered to prospective settlers at such prices plus sufficient commission to cover overhead expenses only.

Every sale contract should be between vendor and purchaser, the Government merely acting as agent. In case of re-sale, the land must be offered

* Annual Rept., 1927, p. 20.

through the same agency for a certain period of years until the project is safely on its way to prosperous settlement.

The Department of the Interior is to be congratulated on its efforts to curb speculation on its recently authorized projects.

PATERNALISTIC AID

It has been recommended that the water users on Federal projects should be further aided by advancing loans, leveling lands, etc. It was the judgment of the Committee that if such paternalistic system is to be extended it should be through local organizations, such as possibly the State or county where there would be a more direct interest in the return of funds and where neighbors would pass on the ability and intent of the borrower to make returns following lines perhaps as adopted by the Farm Loan Banks.

THE ACCOMPLISHMENTS OF THE RECLAMATION SERVICE

The Reclamation Service has constructed 120 dams, built 16 000 miles of canals, laid 3 700 000 ft. of pipe line, and has excavated 252 000 000 yd. of materials in the performance of its duties. In 1925, the estimated crop values produced on these projects was \$77 608 880 on a total area of 1 432 155 acres. The engineers who have been connected with this work may justly regard it with pride. The report that the Committee has presented is not a criticism of this service either past or present. The policies have been fixed by acts of Congress and not by the engineers and executives. This report is intended to be a review of the operation of these policies and an expression on the part of those who are interested and somewhat acquainted with the results.

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A NATIONAL RECLAMATION POLICY:

ECONOMIC ASPECTS OF FEDERAL RECLAMATION*

By ELWOOD MEAD,† M. Am. Soc. C. E.

The report on a National Reclamation Policy prepared by a Committee of the Irrigation Division deals with a problem on which Government officials charged with the Reclamation work may quite properly hold other and contrary views to those expressed. This paper comments on the report as from this angle and, therefore, may be considered as voicing the essentials of another side of the question.

A COMPLEX UNDERTAKING

The reclamation of arid land by irrigation is not a single or a simple undertaking. It involves the construction of works, which is engineering. It requires the settlement and cultivation of land, which is economics. It is not a success unless it creates communities of happy, prosperous homes, which is a social problem. The report‡ under consideration was prepared by engineers. If it had been prepared by economists more attention would have been given to settlement and the creation of conditions which would enable the money spent on construction to be repaid. If it had been written by settlers more would have been said about their needs. Investigations and discussions like those inaugurated by the Society, therefore, will promote an understanding of reclamation and make future development even more valuable.

The influence of Federal reclamation on agricultural development has varied widely in different States. It has been least in California and greatest in Idaho and Arizona. The rich and populous irrigation districts of these two latter States are the creation of Federal works. Outside of California important irrigation works of the future will be built by the Government. Costs are too great and needed income will be too long delayed to make such develop-

* Presented at the meeting of the Irrigation Division, San Diego, Calif., October 4, 1928. Written discussion on this paper will be closed in September, 1929.

† Commr. of Reclamation, U. S. Dept. of the Interior, Washington, D. C.

‡ Proceedings, Am. Soc. C. E., September, 1928, Papers and Discussions, p. 2097.

ment attractive to private enterprise. Government projects must continue to be subsidized either by not requiring interest on construction costs, as at present, or by the Government paying a part of the cost, as is proposed in the report.

The present income for building Federal irrigation works comes from four sources: (1) Payments by water users; (2) payments for power; (3) income from sales of public lands; and (4) a percentage of the income from oil leases. The first two are increasing; the last two are diminishing. The total is about \$10 000 000 per year.

The works being built are large and costly. Their completion will require several years. Farm development must await the water supply; hence, there is small prospect of any material increase in irrigated acreage during the next decade. No contracts for new construction can be made until approved by both the Secretary of the Interior and the President. This operates as a further check on rapid development.

THE SECOND STAGE IN RECLAMATION

There is a gap in reclamation between the completion of canals and the use of water in irrigation. The first step in reclamation is to provide water. The second is to bring it into use. This requires settlers for the uncleared, unleveled land. It requires preparing that land for irrigated culture, erecting farm buildings, and growing crops on soil baked for centuries. The cost, the hardships, and the anxiety of this second step, were always greater than was realized or admitted and the cost is now two or three times what it was fifteen years ago—say, in 1913. An irrigation canal with unpeopled farms below it is a liability, not an asset. Income and the benefits of reclamation are realized only when the second stage of reclamation is completed.

Other obstacles to carrying out settlement and farm development have become more serious in recent years. The pioneering spirit which led settlers to do the difficult and unremunerative work of clearing and leveling the land is gone. The open country no longer appeals as it once did. The opportunities of other industries are much broader. The cost of changing raw land into farms is now so heavy that money or credit is usually needed to supplement the settler's meager capital. Economic surveys of developing reclamation projects made by committees which included practical irrigators, economic experts from agricultural colleges, and representatives of the Bureau of Reclamation show that from \$5 000 to \$10 000 must be spent to provide the permanent improvements and equipment of an 80-acre farm.

The percentage of home seekers with capital enough to improve and equip their farms without borrowing is very small and the number willing to invest their capital in a development of this character is still smaller. They can get more for their money by buying improved farms in established districts. The settlement and development of unimproved, unpeopled areas depends largely on tenant farmers, to whom the lure of ownership is strong, and to the sons of farmers. Such applicants rarely have more than \$2 000 to \$3 000, and settlers with less than \$5 000 capital will need to borrow money to make

their farms produce a living income. There are few projects where this money can be borrowed on terms which the farmer can meet, if it can be borrowed at all. On nearly all developing projects loans are for short periods, with interest rates which are higher than agriculture can stand.

On four projects, which have been in operation for more than ten years, those who have improved farms are prospering; but more than one-half the land is unirrigated, and 800 more settlers are needed to bring all the land in these projects under cultivation. If one-half this number of settlers could be secured it would insure the economic solvency of the Government works and the payment of the Government's debt. These settlers could be secured if there were some agency to loan money needed to erect inexpensive farm buildings and prepare the land for irrigation. They cannot be secured without some credit aid not now provided. Short-time loans to buy live stock can be secured, but money for permanent development is not available.

This credit phase of settlement has been emphasized because it is a stumbling-block in the way of success. It is also a menace to the solvency of works now building or to be built. On one project, where the irrigation works will cost \$11 000 000, there is no anxiety about the payment of construction costs on more than one-half the area. The farms are improved, the land has been prepared for irrigation. The owners have contracted to pay full construction costs on their entire area; cultivation and use of the water are assured. On the unimproved, unpeopled part of this project, however, the surface of the land is uneven and covered with brush or with second growth timber. To prepare this land for irrigation, and to provide the necessary buildings and equipment, will cost from \$75 to \$150 per acre. If this money could be advanced and payment could be spread over 20 years, with 5 or 6% interest, buyers could be secured; but they would be reluctant to take on a clearing and leveling job for which they would have to provide all the money.

This project presents new economic problems, the solution of which needs new policies and new laws. Some of the land belongs to the State, some to a railroad company, some to individuals, and some to the United States. If these owners act together and the preparation of the land for cultivation is put into competent hands, and farm boundaries are fixed to agree with the topography of the country, the land can be made ready for cultivation in less time, at less cost, and with infinitely better results than can be accomplished by leaving this to the action of the separate owners. Each will wait on the others, development will be delayed, and money will be lost. The heaviest loser, of course, would be the Federal Government.

On another project the irrigation works being built have an estimated cost of \$18 000 000. A considerable part of the land under the project is now being irrigated from pumps. The cost of pumping has so increased in recent years that it is now greater than irrigators can afford to pay. A gravity supply will be much cheaper and will therefore avert failure of settlers and bankrupt communities. There is no anxiety about water payments where the lands are settled and improved. Payments will begin, and the water will be used as soon as it is available; but there is a financial problem on 70 000 acres of unleveled, uncleared, and unoccupied land which forms a part of the

scheme. The owners of this land are widely scattered. Hardly any of them expect to become irrigators. They wish to sell. There is little danger of inflated prices. The land has been appraised and the owners have agreed to sell at the appraised price, which is nominal. Cheap land, however, does not insure settlement and cultivation. Here, as in the other case referred to, credit and co-ordinated action are necessary. Part of the land is owned by the State, part by purchasers of State land grants, and part by men who acquired it under the homestead, grazing, and similar acts. A small fraction is owned by the United States. If it is necessary to wait for these independent agencies to improve and develop these farms or find settlers to do it, it will be many years before the water made available is used and before construction costs are returned to the Reclamation Fund.

Anxiety in regard to the financial outlook is met by the statement that the Government is protected in the repayment of its expenditure by district contract. It is true that all the lands in these projects are obligated to pay the entire construction cost and this obligation is a first lien on all the land, but it would impose an undue burden on the developed lands to require them to pay the charges against the undeveloped lands. A part of the project could not carry the cost of the whole. As a practical matter, solvency and settlement are closely related.

The measures for aiding settlers in farm development, suggested in the Committee's report, are entirely inadequate. State aid has been sought. Legislation to require this has been considered in Congressional committees, but investigation showed that some States lack the means to extend this aid, some are prevented from doing so by Constitutional prohibitions, and in every State there is small prospect of political support for this co-operation. The Federal Land Bank has not been, nor is it likely to be, of any assistance in changing raw land into farms. It does not loan money except on income-producing farms, and that means loans are withheld until after the problems of settlement and farm development have been solved.

There is great reluctance in Congress to do more than the Government now does. The reasons for this reluctance are not to be ignored, and it is desirable that further action be avoided and that settlers be found with money enough to make their own improvements and buy their own equipment. There appears to be little hope of this on some of the older projects or on some of those now building. It is the writer's conviction that one of two things should be done: (1) Either provide aid in carrying out the second stage of reclamation; or (2) quit building canals to irrigate unimproved land.

The latest economic report on settlement and farm development recommends that the Government purchase all privately owned, unimproved land on a project before construction begins. If this were done, it would forestall land speculation, enable roads and ditches to be laid out to meet the needs of irrigation, and prevent the imposition of heavy State and county taxes before development is completed. Whether this action is taken, it is desirable that the States be more co-operative than in the past. The rule now is to raise taxes as soon as construction begins. This imposes a burden on the owners of partly improved farms that makes payment of construction costs

difficult and sometimes impossible. The heavy tax burden is one of the arguments used for political action in writing off or postponing payment of construction costs.

A bill now (1928) before Congress is intended to provide a laboratory test of aid and direction in settlement. It authorizes the appropriation of \$500 000 from the Reclamation Fund. It limits the expenditure on any single farm to \$3 000 and restricts loans to provide for the erection of farm buildings and the preparation of land for irrigation, the money thus advanced to be secured by a first mortgage on the land and to be repaid with interest in installments extending over twenty years. If anything is done it should be done with care. If the \$500 000 is authorized, not more than \$100 000 should be appropriated in any one year, and no further authorization should be made until at least one-half these advances has been returned to the Reclamation Fund. This might be arranged through loans to settlers from the Federal Land Bank. Improvements made from these advances would make Federal Land Bank loans possible.

There is also a fear that lending money to settlers would lessen their initiative and self-reliance and tend to make them leaners on the Government. That objection was made to the Federal Land Bank legislation. The same objection applies with greater force to building canals by the Government. It might be said, "Let settlers build their own canals and reservoirs." The answer is that this is impossible because of their cost. The high cost of changing raw land into farms makes it impossible also for worthy, industrious home seekers to get started without aid. There is a kind of aid which does undermine independence and self-reliance, and another kind which strengthens hope and inspires effort. That is what wisely directed credit in farm development would do.

ACHIEVEMENTS OF FEDERAL RECLAMATION IN COMPLETING PRIVATE DEVELOPMENT

It is well to emphasize the gratifying social and economic results which have followed the Government's activities in supplementing or completing district or private development. By taking over the canals of Salt River Valley and building the Roosevelt Reservoir, Federal reclamation rescued discouraged and helpless irrigators and made Salt River Valley, Arizona, one of the most prosperous irrigated sections of the country and a great economic asset of the State. Without similar action the Carlsbad Project in New Mexico would now be only a memory.

In the Weber Valley, in Utah, a reservoir is now being built to furnish water for late irrigation to intensively cultivated land. It will more than double the value of the crops grown. This additional water supply may be likened in value to the water which puts out a fire. It will save disastrous losses in dry seasons and make possible greater profits in all seasons. There is no worry about the payment of construction costs. The water will be used and the payments made according to contract.

The Government has built two reservoirs on Snake River, one in Wyoming and one in Idaho. It is difficult to state adequately the contribution

this has made to the success of irrigated farming in Idaho. Only the Federal Government could have co-ordinated the different interests necessary to carry out a scheme extending over State boundaries. A part of this stored water is being used to give an additional supply to 80 000 acres of land first developed and settled as a State irrigation district enterprise. After farms had been improved and prosperous towns had grown up, it was found that the water supply was inadequate, and ruinous losses from failure of crops were suffered in dry seasons. Towns were shrinking in population and business and farms were being abandoned. The Government, by providing what was impossible to local effort, has restored confidence and prosperity. In the Snake River development there is no uncertainty about the repayment of Government cost. A part was repaid in advance by the water users.

The improvement of an old development and the complete utilization of the resources in land and water through an entire drainage area, are being carried out in the Yakima Valley, Washington. This kind of development is specially suited to the Federal Government. It is solvent and beneficent and there is a broad field for its extension.

The passage of the Adjustment Act in 1926, inaugurated a new era in Federal reclamation. The necessity for this legislation grew out of the hard times which followed the World War. It made it impossible for farmers on some projects to meet their payments. When owners of improved farms in Iowa and Illinois were being sold out, farmers on partly improved farms, under reclamation canals, found their expenses greater than their incomes. It was impossible for them to recover under existing contracts. Congress recognized this situation and passed an Act under which the annual payments of indebtedness in some districts were extended from 20 to 30 and 40 years. The land in all projects was classified in accordance with scientific soil surveys; payments on unproductive lands were cancelled; land injured by seepage, or which from other causes would not at the time grow profitable crops, was given temporary relief from payments. The loss to the Government from this action has been greatly exaggerated. The permanent loss was estimated at \$14 667 965. Some of this will be recovered. The temporary loss was fixed at \$12 788 406. There are hopeful indications that a large part of this will be paid.

Delinquent payments were added to the construction debt. The Reclamation Bureau was authorized to employ economic experts and practical advisers. The foolish idea that practically every one can succeed as an irrigation farmer was discarded and provision was made for examining home seekers by a local board. This has proved a protection to the inexperienced and over-sanguine and is giving reclamation a fair chance to show its value. The Secretary of the Interior has required settlers to have not less than \$2 000 in money or equipment and this leads them to investigate how much developing a farm will cost and to plan their operations with more care.

This Act authorizes the transfer of projects or parts of projects to the water users. Eighteen districts under ten projects have been turned over to local control in the period 1926-28. Where this is done the Government is relieved from any further expense for operation and maintenance. These

changes have promoted co-operation and good feeling between water users and the Government. They have improved the morale of projects and helped to increase construction payments. In 1927 these payments were more than \$1 000 000 greater than in any previous year. Delinquencies once so alarming have almost disappeared. Farm practices are improving. More valuable crops are being grown. There are more acres of sugar-beets, more dairy herds, more farm flocks of sheep, more poultry, and more market gardens. In 1927, the crops produced on the 2 504 046 acres irrigated from Federal works were worth \$133 207 210, which is an average of \$53 per acre, or two and one-half times the average acre value of crops in the United States. The value of this one crop almost equals the entire debt of settlers to the Government. Two such crops will be worth more than the Government has spent on reclamation since the Act was passed. Another crop of equal value is expected this year (1928).

To complete existing projects in accordance with the 10-year program submitted by the Secretary of the Interior to the President and Congress in 1926 will require about \$100 000 000. As the average income of the Reclamation Fund is about \$10 000 000 per year, there need be no nervousness about Federal reclamation increasing the agricultural surplus during the next decade. The endorsement in the Committee's report of this conservative plan of development and of the other economic changes wrought by the Adjustment Act is greatly appreciated.

In its limited field reclamation is one of the most difficult activities of the Government. The Act requires that all money spent shall be repaid. That is impossible, although the Reclamation Bureau has striven to live up to it. Each new project presents new conditions and requires the overcoming of new obstacles. Dams have to be built in remote localities. The suitability of crops to soils has to be tested. Communities have to be organized and markets established. Reclamation has led to a creation of wealth in land many times the cost of the works. Its contribution to other industries, to commerce and trade, entitles it to a credit and support it has not always received. From one town on a Federal project where twenty-five years ago there was nothing but range cattle, a single railroad last year did a business of \$800 000. The indirect benefits from reclamation include help given in solving the problems of soils and climate, improving irrigation practices, founding rural communities which are an economic and social strength to the nation, and creating a wealth in land many times the outlay for works.

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ANALYSIS OF ARCH DAMS BY THE TRIAL LOAD METHOD

Discussion*

By C. H. HOWELL, M. Am. Soc. C. E.†

C. H. HOWELL,‡ M. Am. Soc. C. E. (by letter).§—The interest shown in this paper is appreciated. Mr. Jakobsen states|| “the authors assume that the arch dam is a continuous structure, not divided by contraction joints or cracks” and “to insure freedom from shrinkage cracks, the authors state that openings will be left in the dam and that these will be filled in the coldest weather.”

The first statement is not quite correct. One of the fundamental concepts of the trial-load method is the assumption of joints both horizontal and vertical. These joints are considered incapable of transmitting tensile stresses. In this respect this method of analysis differs from others. The writers did not state that closing plugs would be used to “insure freedom” from shrinkage cracks. The statement in the paper¶ is “to compensate, to some degree, for the shrinkage of the concrete.” The extent of compensation depends on the temperature of the dam at the time of closure, the age of the previously poured concrete, and the number of plugs.

The writers had no idea of suggesting that the plugs would completely offset shrinkage. Mr. Jakobsen states** that “in time when more is known about shrinkage, it can be (and, of course, should be) included in the calculations”. This sentence expresses exactly the thought of the writers.

* Discussion of the paper by C. H. Howell, M. Am. Soc. C. E., and the late A. C. Jaquith, Esq., continued from November, 1928, *Proceedings*.

† Author's closure.

‡ Chf. Engr., J. G. White Eng. Corp., S. en C., City of Mexico, Mexico.

§ Received by the Secretary, February 28, 1929.

|| *Proceedings*, Am. Soc. C. E., April, 1928, Papers and Discussions, p. 1311.

¶ *Loc. cit.*, January, 1928, Papers and Discussions, p. 77.

** *Loc. cit.*, April, 1928, Papers and Discussions, p. 1312.

Mr. Jakobsen calls attention* to the fact that on diagrams in Figs. 5 and 8 no load is shown on the arches at the abutments. He properly questions the correctness of this, as does Professor Cain.† Fig. 59 shows final load diagrams for two Horse Mesa arches. It is apparent that, at the abutments, the arch loads vary within wide limits, as stated by Mr. Jakobsen and Professor Cain. Figs. 5 and 8 are for the earliest studies; Fig. 59 was omitted from the paper in an effort to condense it.

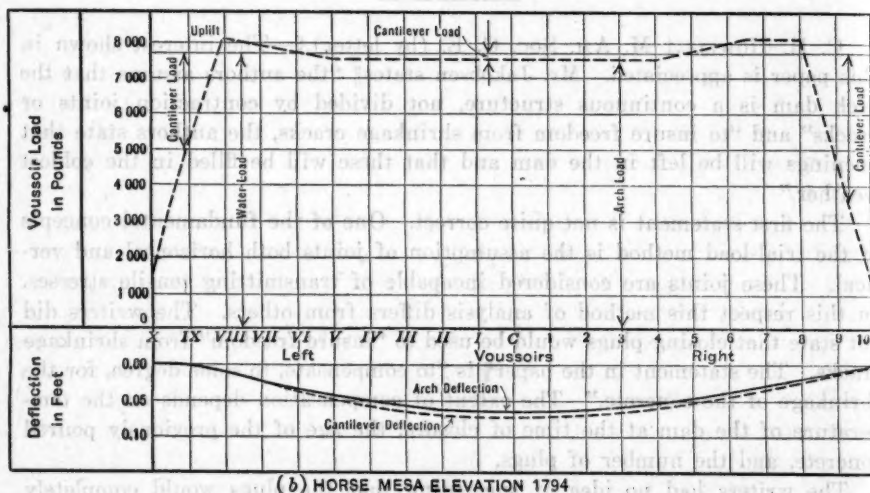
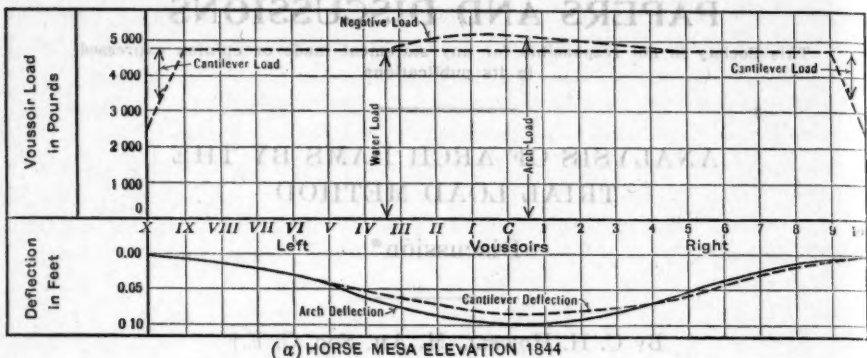


FIG. 59.—FINAL LOAD DIAGRAMS FOR TWO ARCHES, HORSE MESA DAM.

The method of determining the proportional distribution at the abutments suggested by Mr. Jakobsen, is not quite general. For example, let Fig. 60 (a) represent a part of the profile where a sudden change in slope occurs. Consider three equal loads, p_1 , p_2 and p_3 . Using Mr. Jakobsen's notation and method, the distribution of shear stress for p_1 (see Fig. 60 (b)) is expressed by,

$$\frac{p_1 - c}{c} = \frac{\sin 30^\circ}{\cos 30^\circ} = \frac{0.5000}{0.866} = 0.577 \dots \dots \dots (184)$$

* *Proceedings, Am. Soc. C. E., April, 1928, Papers and Discussions, p. 1312.*

† *Loc. cit., August, 1928, Papers and Discussions, p. 1894.*

for p_2 (see Fig. 60 (c)),

$$\frac{p_2 - c}{c} = \frac{\sin 30^\circ}{\cos 60^\circ} = \frac{0.500}{0.500} = 1.00 \dots \dots \dots (185)$$

and for p_3 (see Fig. 60 (d)),

$$\frac{p_3 - c}{c} = \frac{\sin 60^\circ}{\cos 60^\circ} = \frac{0.866}{0.500} = 1.732 \dots \dots \dots (186)$$

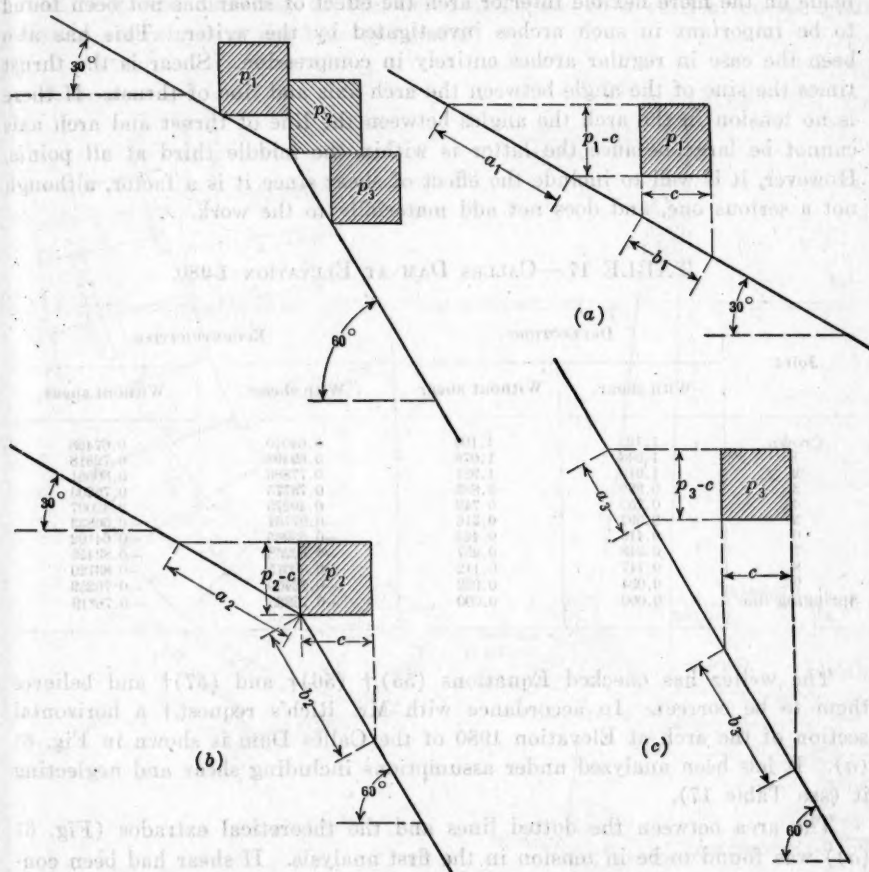


FIG. 60.

Because they are very close together, the loads must have approximately the same distribution and not the wide variation indicated in Equations (184), (185), and (186).

The writer thinks that Mr. Jakobsen* misunderstands the application of Fig. 15. In computing the secondary arches, a new center line is drawn through the middle of the uncracked part of the original arch. The new analysis is then made on this new center line. Its length is divided equally

* *Proceedings, Am. Soc. C. E., April, 1928, Papers and Discussions, p. 1314.*

and the moments of inertia computed on sections normal to it. No oblique sections are used. The lines in Fig. 15 are intended to show the division of the water load into portions tributary to the divisions of the axis.

Mr. Rich,* as well as others, states that in short thick arches the deflections caused by shear are appreciable and should be considered. When such arches are assumed to be capable of resisting tension the effect of shear is considerable. However, if the area affected by tension is excluded and if the analysis is made on the more flexible interior arch the effect of shear has not been found to be important in such arches investigated by the writer. This has also been the case in regular arches entirely in compression. Shear is the thrust times the sine of the angle between the arch axis and line of thrust. If there is no tension in the arch the angles between the line of thrust and arch axis cannot be large because the latter is within the middle third at all points. However, it is well to include the effect of shear since it is a factor, although not a serious one, and does not add materially to the work.

TABLE 17.—CALLES DAM AT ELEVATION 1980.

Joint.	DEFLECTIONS.		ECCENTRICITIES.	
	With shear.	Without shear.	With shear.	Without shear.
Crown	1.121	1.109	0.64010	0.67438
1	1.043	1.078	0.69498	0.72818
2	1.018	1.981	0.77886	0.80661
3	0.899	0.885	0.73775	0.76203
4	0.757	0.742	0.40275	0.42007
5	0.591	0.516	-0.07701	-0.06833
6	0.416	0.423	-0.63963	-0.64102
7	0.248	0.257	-0.82268	-0.83456
8	0.117	0.112	-0.80675	-0.83120
9	0.084	0.082	-0.72400	-0.76259
Springing line	0.000	0.000	-0.73385	-0.78819

The writer has checked Equations (55),† (56)† and (57)† and believes them to be correct. In accordance with Mr. Rich's request,† a horizontal section of the arch at Elevation 1980 of the Calles Dam is shown in Fig. 61 (a). It has been analyzed under assumptions including shear and neglecting it (see Table 17).

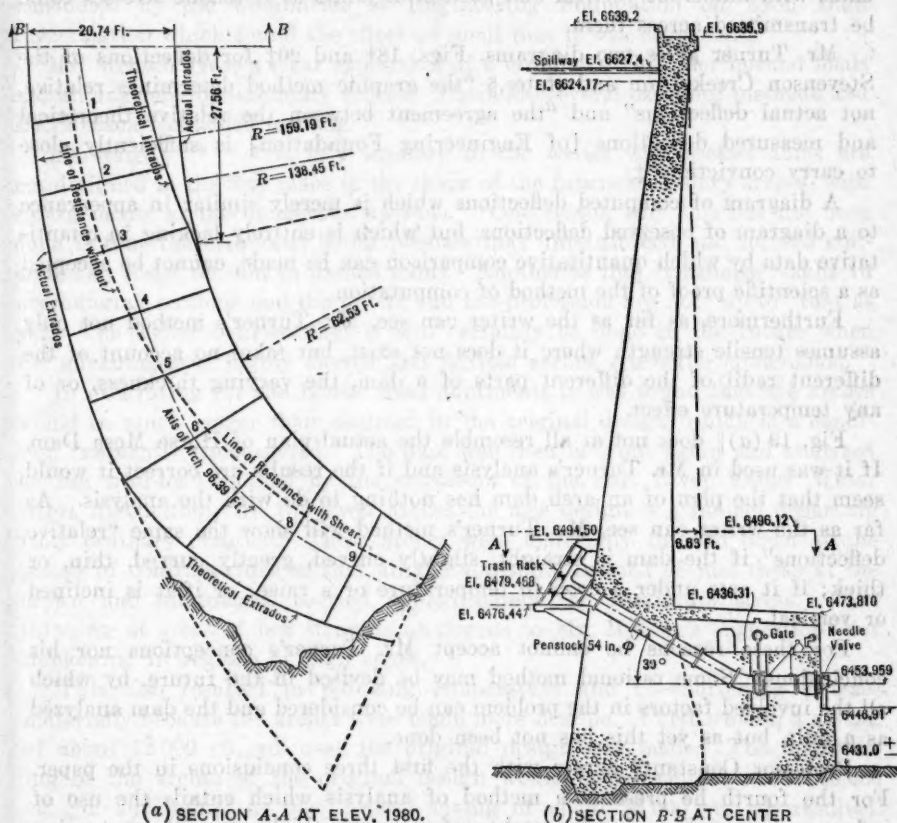
The area between the dotted lines and the theoretical extrados (Fig. 61 (a)) was found to be in tension in the first analysis. If shear had been considered in connection with tensile stresses the effect would have been considerable because the thrust line was far outside the middle third and formed large angles with the original axis.

Analysis of the "interior arch" (between the theoretical extrados and theoretical intrados) which is entirely in compression, shows that the difference between the positions of the thrust lines with shear and without it is too small to be platted. The computed deflections and eccentricities as shown in Table 17 are negligible.

* *Proceedings, Am. Soc. C. E.*, April, 1928, Papers and Discussions, p. 1315.

† *Loc. cit.*, p. 1317.

Mr. Turner condemns* the conception of arches alone and arches combined with cantilevers in analyzing curved dams. He proposes as the correct method the analysis of an arch dam as if it were a curved plate. In this position he is at variance with all others contributing to the discussion and most, if not all, engineers who have written on the subject. However, "engineering is not a matter of majorities." It may be that Mr. Turner is right. Of this, however, the writer is yet to be convinced. He cannot accept the statement† that "a curved dam may be considered analogous to a vertical curved plate rigidly restrained at the base and ends."



It seems to the writer that the only force that "fixes" a dam to its base in a vertical direction is its own weight plus whatever vertical water load it may carry. Furthermore, the only force that "fixes" a curved dam to the abutments in a horizontal direction is the arch thrust. If the resultant of forces is outside the middle third in either direction the dam will tend to rotate in that direction. It cannot then be considered as fixed. All dams

* *Proceedings, Am. Soc. C. E.*, May, 1928, Papers and Discussions, p. 1585.

† *Loc. cit.*, p. 1586.

known to the writer have construction joints, both vertical and horizontal. The entire base of the dam is a construction joint. The writer knows of no evidence to the effect that tension can be transmitted across ordinary construction joints unless reinforcement is used.

The student in the engineering school mentioned by Mr. Turner* would be considered deficient in knowledge of the ordinary theory of concrete design and practice if he proposed to take advantage of the tensile strength of concrete in the design of a jointless concrete beam. Yet some post-graduate engineers with years of experience undertake to analyze unreinforced concrete dams with numerous vertical and horizontal joints as if tensile stresses could be transmitted across them.

Mr. Turner gives two diagrams, Figs. 18† and 20‡ for deflections of the Stevenson Creek Dam and states,§ "the graphic method determines relative, not actual deflections" and "the agreement between the relative theoretical and measured deflections [of Engineering Foundation] is sufficiently close to carry conviction."‡

A diagram of computed deflections which is merely similar in appearance to a diagram of observed deflections, but which is entirely lacking in quantitative data by which quantitative comparison can be made, cannot be accepted as a scientific proof of the method of computation.

Furthermore, as far as the writer can see, Mr. Turner's method not only assumes tensile strength where it does not exist, but takes no account of the different radii of the different parts of a dam, the varying thickness, or of any temperature effect.

Fig. 19 (a)|| does not at all resemble the actual plan of Horse Mesa Dam. If it was used in Mr. Turner's analysis and if the results are correct it would seem that the plan of an arch dam has nothing to do with the analysis. As far as the writer can see, Mr. Turner's method will show the same "relative deflections" if the dam is straight, slightly curved, greatly curved, thin, or thick; if it acts under a drop in temperature or a raise; or if it is inclined or vertical.

For these reasons he cannot accept Mr. Turner's conceptions nor his conclusions. Some rational method may be devised in the future, by which all the involved factors in the problem can be considered and the dam analyzed as a unit, but as yet this has not been done.

Professor Constant¶ agrees with the first three conclusions in the paper. For the fourth he presents a method of analysis which entails the use of influence lines. For circular arches of constant thickness without tension this method should be more rapid than that given in the paper.

In all the dams thus far studied by the writer it has been thought advisable to increase the abutment thickness either because of pressure in uncracked arches or to provide for the formation of adequate interior arches

* *Proceedings, Am. Soc. C. E.*, May, 1928, Papers and Discussions, p. 1592.

† *Loc. cit.*, p. 1587.

‡ *Loc. cit.*, p. 1591.

§ *Loc. cit.*, p. 1589.

|| *Loc. cit.*, p. 1588.

¶ *Loc. cit.*, p. 1593.

where tension was indicated. In the case of non-circular or irregular shaped arches, the shapes of which change with each trial computation, the influence-line method may not be so advantageous.

Mr. Hanna,* as did Mr. Rich, notes the neglect of shear and develops formulas for including it. He also calls attention to the fact that the arches do not deflect radially except at the crown and that the lateral cantilevers are affected by the torsional force required to twist them to the deflected arch, and *vice versa*.

Dr. Vogt† states that this effect is not appreciable. This matter was also considered by the Committee of Engineering Foundation on Arch Dam Investigation which found the effect so small that it was neglected.‡

Mr. Mensch gives a clear exposition of a circular arch under normal loads and an analysis for secondary arches. There are several excellent methods and Mr. Mensch has added to them.

He states,§ “* * * it is a mystery to the writer why arched dams are not designed in the first place in the shape of the future secondary arches, with enlargements at the dangerous sections.” One reason why this has not been done is that the erroneous belief persists that unreinforced and jointed concrete can resist tension in arched dams. Another is that it usually results in overhanging sections and the public and the profession are not “up to” that as yet. The writer believes that it will certainly be done in the future when the advantages of highly curved and flexible arches are better understood.

In excavating for the Horse Mesa abutments it was found that the arches would be much longer than assumed in the original design, which is a condition frequently encountered. The dam was then laid out again and analyzed by the trial-load method by the engineers of the Salt River Valley Water Users Association. In the lower arches the new design is a radical departure from ordinary practice. These arches were materially thinned down at the points of contraflexure so that all the voussoirs, instead of just those at the crown and abutments, carried approximately the maximum stress. This thinning at areas of low stress is analogous to Mr. Mensch's suggestion|| for thickening in regions of high stress.

This also resulted in reducing temperature and rib-shortening stresses materially because the arches were made more flexible. Furthermore, a saving of about 12 000 cu. yd. over the original design was made. The lengthening of the arches of the original design would have required about 13 000 cu. yd. additional, so that a total saving of about 25 000 cu. yd. resulted. Moreover, the arches are more evenly stressed than in the original design.

In the Calles Dam, designed by Julian Hinds, M. Am. Soc. C. E., which has been completed recently, a part of the concrete in the tension areas at the abutments of the lower arches was omitted, as shown in Fig. 61 (a). The rock was simply left in place and the upper arches, while overhanging the lower

* *Proceedings*, Am. Soc. C. E., May, 1928, Papers and Discussions, p. 1598.

† *Loc. cit.*, November, 1928, Papers and Discussions, p. 2516.

‡ *Loc. cit.*, May, 1928, Pt. 3, p. 153.

§ *Loc. cit.*, August, 1928, Papers and Discussions, p. 1886.

|| *Loc. cit.*, p. 1880.

ones, were actually supported on the undisturbed rock. This resulted in an appreciable saving.

Savings similar to those made at the Horse Mesa and Calles Dams can and will be made in the future by omitting the unnecessary parts of the arches. The writer believes it is practical to support the overhang, if necessary, on piers or webs. He is in complete agreement with Mr. Mensch* that it would be more satisfactory if cantilever action could be eliminated. This, however, cannot logically be done with vertical dams. A section of any vertical wall will carry some water load. This cannot be reasonably neglected in considering the arch loads. Vertical elements in inclined dams can carry no water loads; the arches take it all. The problem is thus simplified.

The writer believes that inclined arch dams with tension areas eliminated and with thinner sections in regions of low stress or thicker sections in regions of high stress, will be common in the future.

Professor Cain† points out the fact that the method as described neglected the friction between the arches, and that this neglected factor contributes to their strength. Dr. Vogt‡ also goes into this matter of tangential resistance in more detail and shows that in wide dam sites it is an appreciable factor.

It is pleasing to the writer that Professor Cain§ so strongly emphasizes the fact that tension cannot be transmitted across cracks and ordinary construction joints. This error has been most persistently retained in other methods of analysis. The statements of Professor Cain will go far in eliminating this fallacy in the future.

The writer is indebted to Messrs. Savage and Houk for their discussion.|| It clearly explains the latest development of the trial-load method. He believes their discussion will extend the use of this method generally.

Returning for the moment to the effect of shear, attention may be called to Fig. 50.¶ This arch has appreciable areas in tension. Both lines of thrust (with and without shear) are outside the middle third. If the shape of the arch were modified by increasing the abutment thickness so as to permit an interior arch to form, which would be entirely in compression, the line of thrust would be within the middle third. It would be more nearly parallel to the center line. Therefore, the effect of shear would be materially less than that indicated in Fig. 50.

It is reassuring to know that in the case of the Gibson Dam the refined methods indicated no substantial changes in the results given by the original computation. Horse Mesa is the only dam designed by the trial-load method, as described in the paper, which is built and under full load. The water surface at Calles is about 50 ft. from the top. Horse Mesa is one of the very highest of dams and probably the thinnest, for its height, in the world. It shows no sign of distress and has carried its full load for several years.

* *Proceedings*, Am. Soc. C. E., August, 1928, Papers and Discussions, p. 1891.

† *Loc. cit.*, p. 1894.

‡ *Loc. cit.*, November, 1928, Papers and Discussions, p. 2521.

§ *Loc. cit.*, August, 1928, Papers and Discussions, p. 1897.

|| *Loc. cit.*, October, 1928, Papers and Discussions, p. 2317.

¶ *Loc. cit.*, p. 2320.

The Stevenson Creek Dam was analyzed by the trial-load method soon after it was first proposed. In this analysis (see Fig. 62) the effect of shear was not included in either arches or cantilevers. It was assumed that the concrete was incapable of taking tension; that the modulus of elasticity was 2 000 000; and that there was a temperature drop of 20° Fahr. The load tests on the actual dam were conducted in such a way that they did not include appreciable temperature changes.

The maximum arch stress computed by this analysis was 1 600 lb. per sq. in. The maximum given in the Report of the Committee of Engineering Foundation on Arch Dam Investigation* is about 1 100 lb. A change in temperature between 8° and 3.5° gave a stress of about 100 lb. per sq. in., or a total of 1 200 as against 1 600 as given by the analysis for a temperature drop of 20 degrees. It is seen that this crude analysis, based on assumptions considerably at variance with conditions at the time of testing, gave conservative results.

Mr. Noetzli† notes that the trial-load method has been used in the design of the Horse Mesa, Gibson, and Owyhee Dams. To this list should be added the Calles Dam, previously mentioned. This is a very thin dam with a slight overhang in the central section, as shown in Fig. 61 (b).

Mr. Noetzli states‡

"* * * it is perhaps indicated that future efforts should be directed toward alterations in the type of structure, such as would eliminate, in part at least, * * *, some of the uncertainties which are inherent in * * * the thick arch dams".

With this statement, the writer is in complete accord.

Dr. Vogt§ is constructive. He points out, as does Mr. Noetzli,† that, once a crack has formed, the occurrence of adjacent cracks in the concrete is not probable. The writer thinks this point is well taken, and that it constitutes an uncertain element in considering "secondary arches". He also believes that removing the material down to the surface of the secondary arch does not impair the strength; it is a distinct advantage in that it makes the analysis more definite, the arches more flexible, and saves material. Attention properly may be called, however, to the fact that most dams are built in comparatively low lifts. Each lift, therefore, is bounded by construction joints. If not reinforced, these joints cannot consistently be assumed to take tension. To the writer's mind this condition justifies the assumption that cracks will occur whenever the resultant is outside the middle third. A gravity section which shows tension at any level will tend to open up at that level. The same action may be expected in the vertical elements in an arch dam.

The proposal mentioned by Dr. Vogt|| to improve arch action by pressure grouting was made¶ several years ago by L. R. Jorgensen, M. Am. Soc. C. E.

* *Proceedings*, Am. Soc. C. E., May, 1928, Pt. 3, p. 141.

† *Loc. cit.*, October, 1928, *Papers and Discussions*, p. 2325.

‡ *Loc. cit.*, p. 2326.

§ *Loc. cit.*, November, 1928, *Papers and Discussions*, p. 2515.

|| *Loc. cit.*, p. 2516.

¶ *Transactions*, Am. Soc. C. E., Vol. LXXXIII (1919-20), p. 316.

It seems to be theoretically correct if it can be practically accomplished, but as far as the writer is aware, it has never been actually tried.

It may be difficult to secure a uniformity of pressure distribution and almost impossible to be sure of it. Experience alone will demonstrate its practicability.

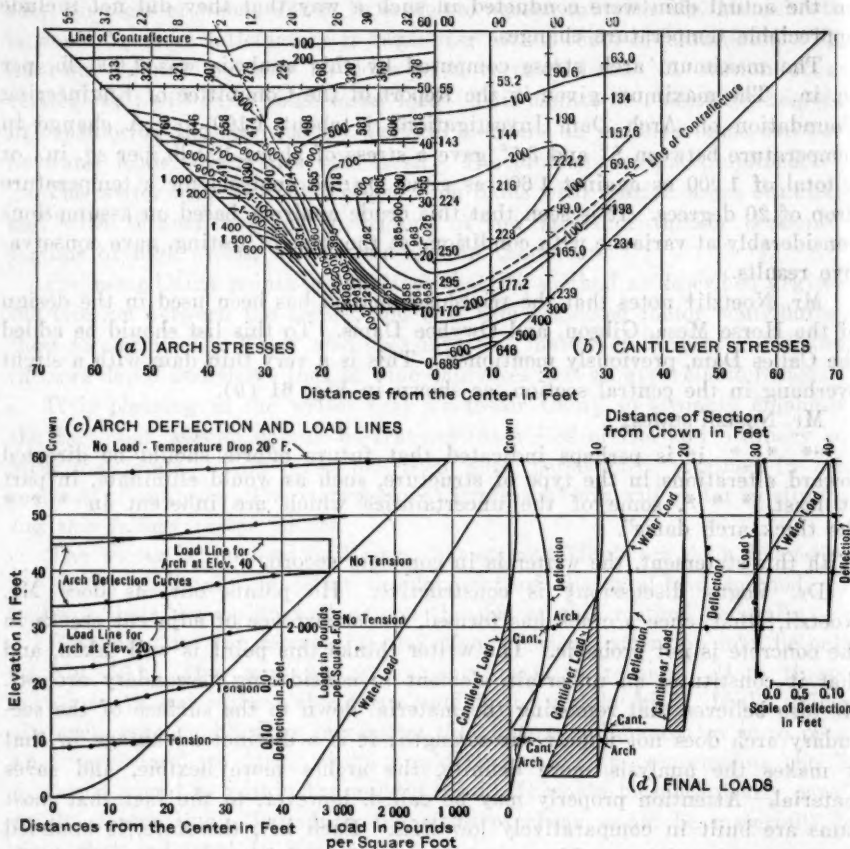


FIG. 62.—ANALYSIS OF STEVENSON CREEK DAM BY THE TRIAL-LOAD METHOD.

During the construction of the Calles Dam consideration was given to using jacks in the gap of the closing plug to compensate in some degree for shrinkage. It was proposed to concrete the jacks in place. The computations indicated that it was quite possible to deflect the dam up stream appreciably by a reasonable number of jacks. Certain mechanical problems arose and the time necessary to solve them prevented this experiment from being completed. The ordinary concrete closing-plug was used. There will be mechanical difficulties in the grouting process also.

The proposal* to cool the dam by circulating water through it is interesting and seems practical if the benefits will warrant the expense.

* *Proceedings, Am. Soc. C. E.*, November, 1928, Papers and Discussions, p. 2516.

Dr. Vogt states* that three additional factors might well be considered. Bending due to lateral expansion, stiffness against torsion (mentioned by Mr. Hanna†), and the stiffness against tangential displacements. The first he believes will usually be negligible. In two cases he found that torsional resistance reduced the stresses slightly and he, therefore, recommends that this factor be neglected except perhaps for very thick dams. He states that resistance against tangential displacements is of considerable importance at wide dam sites, and gives mathematics for an approximate solution, together with the results of an interesting model test to confirm his conclusion.

His statement‡ that multiple-arch dams may possibly be built without buttresses is startling. It is not evident to the writer how the usual multiple-arch section could be deprived of its buttresses, stood up vertically, and at the same time be safe from sliding even if the arch thrusts could be theoretically carried across the various units.

Regarding the cases§ in which gravity abutments failed and the arches stood, it should be stated that one of these dams, the Moyie, was completely reinforced by 20-lb. rails on 2-ft. centers, both horizontal and vertical.|| The other, Lake Lanier, was reinforced for the top 12 ft.§ of its total height of 60 ft.,|| or 20 per cent. At the end of the arch where the abutment failed these 12 ft. comprise about 45% of the height. The reinforcement could not help stiffening the arches against tangential displacement during the failure of the abutments. If the arches had not been reinforced the tangential resistance alone might not have saved them.

Mr. Floris¶ expresses impatience that the writers gave some formulas for analyzing arches. He then gives some of his own.

Summing the results of the discussion it may be said that the first three conclusions in the paper are unquestioned by all but Mr. Turner. As to the fourth, nearly every one has pointed out omissions and suggested improvements in the mathematical treatment. This was expected and is welcomed. It was distinctly stated** in the paper that the method was not considered complete. Several of the omitted factors were specifically mentioned.†† The discussion has shown how tangential resistance and shear effect may now be included. Dr. Vogt has dealt with foundation deformation so that it may also be considered. In time, it seems that it will be possible to include all pertinent factors.

In conclusion it is frankly admitted that at present the trial-load method is tedious and cumbersome. However, it may be stated that a graphic method developed by Mr. Hinds indicates that a complete solution may be made in a fraction of the time now required.

* *Proceedings*, Am. Soc. C. E., November, 1928, Papers and Discussions, p. 2516.

† *Loc. cit.*, May, 1928, Papers and Discussions, p. 1601.

‡ *Loc. cit.*, November, 1928, Papers and Discussions, p. 2523.

§ *Loc. cit.*, p. 2523.

|| *Engineering News-Record*, Vol. 97, 1926, p. 616.

¶ *Proceedings*, Am. Soc. C. E., November, 1928, Papers and Discussions, p. 2532.

** *Loc. cit.*, January, 1928, Papers and Discussions, p. 78.

†† *Loc. cit.*, p. 63.

It is believed that the trial-load method gives a reasonably accurate analysis. The conception of vertical and horizontal elements is supported by the Stevenson Creek tests. In the report of the Committee of Engineering Foundation on Arch Dam Investigations,* it is stated,

"For the most part the directions of the principal strains were approximately vertical and horizontal. This indicates that the division of the dam into imaginary vertical and horizontal elements for purposes of analysis not only serves convenience, but also indicates that the stresses found in this manner will generally be close to the principal stresses."

An inspection of the plans of the dam designed by this method will show that they are economical in material. The discussion has shown that the method determines the maximum stresses conservatively. Further development of the method is expected to result in more accuracy and increased economy both in material and in time of designing.

For small thin dams with large central angles the application of the trial-load method may not be necessary. For such dams the formulas developed by Professor Cain are thought to be adequate. For high thick dams and especially for those with small central angles and irregular profiles the writer thinks the use of the trial-load method is advisable. His thanks are extended to those who discussed this paper.

* *Proceedings, Am. Soc. C. E., May, 1928, Pt. 3, p. 193.*

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FLOOD CONTROL ON THE RIVER PO IN ITALY

Discussion*

By JOHN R. FREEMAN, PAST-PRESIDENT, AM. SOC. C. E.†

JOHN R. FREEMAN,‡ PAST-PRESIDENT, AM. SOC. C. E. (by letter).§—The kindly words of Colonel Townsend|| about the information given in the paper are most welcome, especially his hearty appreciation of the work of the early Italian engineers. These men were the founders of hydraulic science, notwithstanding the fact that it required more than a century of their earnest work to place the square root sign in the formula for the discharge from an orifice, and that a hundred years still later it remained for a French engineer to invent the Chezy formula for stream flow.

During the Middle Ages, Northern Italy was the world's greatest center of universities, and those who governed the country called into service for river regulation and flood protection the foremost professors of physics and mathematics of their time. The writer believes that students in the new schools of hydraulic research at the great engineering colleges of Italy—with the aid of recently established hydraulic laboratories, and the new observations on the River Po—will learn more of controlling these waters economically in the next forty years than has been learned by work on the river alone in the past four hundred years.

Colonel Townsend refers¶ to the fact that the Italian civil engineers are taught the fundamental principles of river hydraulics in the various engineering colleges of that country, while this subject is commonly ignored in American engineering colleges. The writer understands that almost every im-

* Discussion of the paper by John R. Freeman, Past-President, Am. Soc. C. E., continued from November, 1928, *Proceedings*.

† Author's closure.

‡ Cons. Hydr. Engr., Providence, R. I.

§ Received by the Secretary, January 29, 1929.

|| *Proceedings*, Am. Soc. C. E., August, 1928, Papers and Discussions, p. 1997.

¶ *Loc. cit.*, p. 2002.

portant college in Europe has a Professorship in River and Harbor Hydraulics, while there is not yet one in any college on the American Continent.

The fundamental reason for this is that no attractive career opens, even to the brightest student from the foremost engineering college—whichever that might be—if he begins and continues as a United States Assistant Engineer. He can never rise to the top and have his name attached to any important structure (*vide* the experience of the late Alfred Noble, Past-President, Am. Soc. C. E.). Up to the present time the only door opening to such a career is by way of the National Military Academy at West Point. While the eminent Army engineer may retire at the age of 65, with a pension of about \$5 000 per annum, and thereafter carry on a lucrative consulting practice, even the ablest civilian engineer on river and harbor work in the United States must either resign, or work on at about one-half the salary of the West Point graduate until he is 70 years of age, when he will be dismissed, with a pension of only about \$1 000 per annum.

The Army method, which largely makes voucher clerks of the Resident Army Engineer's staff, and after a very few years on any one problem shifts the leading officers to new fields, cannot possibly produce the best results for research on river and harbor improvement; but it is doubtless the best peacetime method of training Army officers for a future war. Army methods assume that any one man can replace any other man of equal grade—and that is a necessity in the field of battle; but the fundamental facts of human nature remain—that probably not one good average engineer out of ten (perhaps only one out of a hundred) has the inborn talent to become a great research man and develop that initiative which advances the science and the art.

Colonel Townsend deprecates the need of hydraulic laboratories for studying river problems.* If he should visit Italy and Germany again he would be amazed, as the writer was, to observe the development of that most useful of aids in experimental research, the Doctrine of Similitude, with the use of small, relatively inexpensive models, which is now placed in the hands of the skilled engineer striving to build better hydraulic turbines, and of those engaged in river and harbor engineering. It is the strongest tool ever invented for advancing the state of the hydraulic arts.

In due course it may revolutionize methods in research on river flow, and tidal action in harbors, just as the naval tank in the hands of Froude, fifty years ago, gave a new departure for problems in ship modeling.

So rapid and recent is this progress in the method of attack on hydraulic problems† that many people have overlooked the remarkable research made in the hydraulic laboratory of the Engineering College at Manchester, England, on the projected creation of a great tidal water power by means of a dam across the estuary of the Severn River. It is interesting to note that a model of the river and bay, 40 miles long, is housed in a room 45 ft. square, and is constructed with such nicety and accuracy of detail that it cost about \$10 000.

* *Proceedings*, Am. Soc. C. E., August, 1928, Papers and Discussions, p. 2002.

† "Hydraulic Laboratory Practice," pub. January 1, 1929, by the Am. Soc. Mech. Engrs., New York, N. Y.

The laboratory idea is developing and spreading so fast in Europe that, within the last year or two, great new hydraulic laboratories have been brought to completion by the Swiss Government at Zurich; by the Netherlands Government at Delft; the Norwegian Government at Trondhjem; and the Czecho-Slovakian Government at Prague. Surely such hydraulic laboratories are not built, as Colonel Townsend suggests,* "for the purpose of rediscovering principles known in Italy for centuries and now taught in the ordinary textbooks of the engineering schools of Europe." It is the firm belief of the writer that tens of millions of dollars have been wasted on river and harbor work in the United States through failure to realize the value of the hydraulic laboratory and the applications of the doctrine of hydraulic similitude.

It is difficult for a stranger in a foreign land unfamiliar with the language to make sure that he correctly understands the data placed before him. Hence the writer values the discussion† of Professor Giandotti particularly because of its acknowledgment that the "paper describes very well the actual conditions". Professor Giandotti emphasizes some of the finer points that the writer missed. Furthermore, the discussion is valuable because of the proof given in Fig. 14‡ that, although the elevations of the river bed fluctuate from year to year, as the volume of sediment is moved along, and that "while they are never the same from year to year, the elevations always oscillate about a line of equilibrium and show practically no appreciable net change."§

The discussion by Mr. Goodrich§ is of interest because of comparisons of conditions along the Po with those along certain waters in Chihli in Northern China. This confirms similar statements once made before the Society by the writer which at first were regarded as unbelievable. The percentage in volume of silt to water brought down by the River Po is surely very moderate as compared with that of some of these Chinese rivers.

Mr. Grunsky's contribution|| is of particular interest in further emphasizing the difference in the problems of flood control presented by rivers in various parts of the United States and of the world. Surely, however, the fundamental laws and principles of erosion, sedimentation, and stream flow, apply to all. The differences mentioned by Mr. Grunsky result simply from different values of the factors in the same general equation, presented by the variable circumstances of the drainage area and the precipitation in the several localities. The writer has been impressed with the differences in sediments, or material shoved along by the current, in rivers with rapid slope leading out from the Alps as compared with those found in the beds of most American rivers in the East. On the other hand, in rivers coming down the western slopes of the Sierras and the Cascade Mountains gravels and cobblestones are carried along as in the Alpine rivers. The mechanical laws are of universal application.

* *Proceedings, Am. Soc. C. E.*, August, 1928, Papers and Discussions, p. 2002.

† *Loc. cit.*, November, 1928, Papers and Discussions, p. 2569.

‡ *Loc. cit.*, p. 2570.

§ *Loc. cit.*, p. 2571.

|| *Loc. cit.*, p. 2576.

Regarding Colonel Slattery's dictum,* that it is "unquestionably cheaper" to maintain the necessary depth by dredging than by spur-dikes and retards, the writer suggests that dredging was long ago developed to the maximum possibilities of efficiency, while spur-dikes and retards of twenty years ago may have fallen very far short of the best and most economical that could be devised by further studies in the laboratory and in the field. The time has come to re-open some of the questions that were considered "settled" forty years ago.

Colonel Slattery well describes the marvelous efficiency with which levees were topped out by sand-bags during the Mississippi flood of 1916. The writer, as a student of river hydraulics, followed the greater flood of 1922 along down the river, and was thrilled by the wonderful rapidity and efficiency with which levees were pieced up and sand-bags placed, and threatened crevasses checked; and he fully believes that no better or more energetic work has been accomplished anywhere in the world, in times of such great emergency. Much of it was done by civilian engineers and chiefs of levee districts. Nevertheless, the work along the Po in the great flood of 1927, by which 30 miles of sand-bag dikes were placed on top of the dikes of the river, which rises very much more rapidly than the Mississippi, is of a similar order of excellence and probably superior in some of the details. The sudden rise of the Po permits no such relatively leisurely preparation in collecting grain-sacks, from a thousand miles away, as is possible along the Lower Mississippi, where the flood height following abnormal rains can be predicted with a fair degree of precision several weeks in advance.

Regarding the fact, that "overflows [along the Mississippi] are not always an unmitigated curse", the writer believes that something can be learned from the system along the River Po, where there are two sets of dikes, one near the river and a higher one farther back, with numerous cross-sections, thus permitting an occasional overflow and enrichment of the broad strip near the river. So far as the writer could learn, the valley floor of land subject to inundation along the Po does not pitch back from the river, or decline in altitude, to any such degree as that along the Mississippi, where, in general, the bottomlands near the river are fully 5 ft. higher than they are a mile back. This condition along the Mississippi is adverse to arranging a system of double dikes as along the Po; but the writer is strongly inclined to believe that thorough study would show that a double dike system in certain parts of the Mississippi delta would give the best possible protection, and by proper design of intakes would permit periodically flooding the intermediate land for restoration of fertility. In contrast to the Po, the dike nearest the Mississippi should be the main line of defense, and the series of secondary dikes, running in straight lines about $\frac{1}{4}$ mile back from the apex of principal bends, should here be regarded as the second line of defense, which would prevent the flood from escaping and flooding, for example, the whole Yazoo Valley from an unsuspected sand boil and midnight crevasse.

* *Proceedings, Am. Soc. C. E., September, 1928, Papers and Discussions, p. 2137.*

Replying to the questions of Mr. Davis,* the writer found that there was, apparently no doubt among Italian engineers that the Po was steadily lengthening its delta, and projecting its load of sediment into the Adriatic, thereby constantly lengthening the river's course.

Dr. de Thierry† who was born in Northern Italy, knows the history, the language, and the work of the Italian engineers. Perhaps his most useful contribution to this discussion is that about the time factor involved in the study and working out of these great river problems, which he suggests is a matter of a century or so; and, that "the Po, like every other river, is, in effect, a living organism, the study of which requires more than the lifetime of one man."

* *Proceedings, Am. Soc. C. E., September, 1928, Papers and Discussions, p. 2141.*

† *Loc. cit., p. 2143.*

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HYDRAULIC STUDIES AND OPERATING RESULTS ON THE MIAMI FLOOD CONTROL SYSTEM

Discussion*

BY CHARLES H. PAUL, M. AM. SOC. C. E.

CHARLES H. PAUL,† M. AM. SOC. C. E. (by letter).‡—Full-scale results, under actual operating conditions, are most valuable to the engineer who is faced with a problem of design. Had information of this kind been available at the time the Miami flood-control project was designed, much time, labor, and expense would have been saved to the District.

There is much of value in this paper which unfortunately cannot be indicated by the title. Some items, especially, should be mentioned, as follows: (1) Actual capacities of large conduits under heads up to 40 ft. or more; (2) actual values of n for improved river channels under various conditions; (3) practical demonstrations of the economical methods of securing the enlargement of cut-off channels by scouring; (4) methods of reducing maintenance costs by gravel plants and low barrier dams; (5) methods of measuring stream flow under high velocities; (6) effect of the hydraulic jump in reducing destructively high velocities; and (7) a most emphatic demonstration of the effect of soil conditions on run-off.

There is a slight difference in the discharge factors of the Englewood and Germantown conduits which, apparently, is hard to account for at this time. The writer believes, however, that when more measurements are available and a more detailed study has been made, this difference will be reduced or explained. As a matter of fact, as it now stands it is not of great importance, except that a logical reason for it is not apparent. Both conduits show a larger actual discharge than was indicated by preliminary estimates. This is not surprising: First, because of the inclination to throw factors of safety

* Discussion on the paper by C. H. Eifert, M. Am. Soc. C. E., continued from January, 1929, *Proceedings*.

† Cons. Engr.; Managing Director, Dayton Industrial Assoc., Dayton, Ohio.

‡ Received by the Secretary, February 13, 1929.

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THE DESIGN OF TALL BUILDING FRAMES TO RESIST WIND

Discussion*

By ALBERT WARD ROSS, JR., JUN. AM. SOC. C. E., AND CLYDE T. MORRIS,
M. AM. SOC. C. E.†

ALBERT WARD ROSS, JR.,‡ JUN. AM. SOC. C. E., and CLYDE T. MORRIS,§ M. AM. SOC. C. E. (by letter).||—The discussions provoked by this paper have been extremely interesting and it is certain that the profession has benefited by the valuable points affecting practical design which have been brought out in them. Another valuable product is the publication of several additional methods of calculation which heretofore had not been generally available.

The writers wish to correct one impression appearing in several of the discussions, namely, that the proposed method claims to be "exact". It certainly should be classed among the so-called "approximate" methods, but it is believed that the results obtained approach nearer to those of the elastic theory in most cases, than other approximate methods which do not take into account the variation in stiffness of members.

The entire problem rests on the validity of the assumptions,¶ and, of these, the one most frequently attacked is that "the wind load is resisted entirely by the steel frame". There can be no question but that the stiffness of the walls and partitions materially assists the steel frame in resisting distortions due to lateral forces; nevertheless, the effects of the Florida hurricane in 1926 prove conclusively that buildings with steel frames properly and adequately proportioned, suffered no structural damage, while others, in which the steel frame was weak, failed. The added strength provided by the walls and partitions is utilized in all building codes and specifications, when the wind

* Discussion on the paper by Albert Ward Ross, Jr., Jun. Am. Soc. C. E., and Clyde T. Morris, M. Am. Soc. C. E., continued from April, 1929, *Proceedings*.

† Authors' closure.

‡ Aircraft Eng. Dept., Ford Motor Co., Dearborn, Mich.

§ Prof. of Structural Eng., Ohio State Univ., Columbus, Ohio.

|| Received by the Secretary, March 28, 1929.

¶ *Proceedings*, Am. Soc. C. E., May, 1928, Papers and Discussions, p. 1397.

loads are specified at from 15 to 30 lb. per sq. ft.; whereas, it is known that a wind velocity of 120 miles per hour will produce pressures as great as 50 lb. per sq. ft. From this, it is evident that, for safety, the structural frame must function as a unit, independent of assistance which it may receive from the "architectural clothing". If the steel frame does so function, and is properly designed, it must do so in accordance with the elastic theory and not any given set of arbitrary assumptions regarding the location of points of contraflexure and distribution of shears or direct stresses.

The assumptions of the slope deflection method as developed by Wilson and Maney* are believed to be adequate to produce results closely in accord with the elastic theory, but the labor involved is great. Hence, the attempt of the writers to develop empirical curves for determining for any given set of conditions the error of an approximate method, closely resembling the method developed by Albert Smith, M. Am. Soc. C. E.† As ordinarily constructed, the walls and partitions are not sufficiently bonded to the steel work to cause the two to act together as a unit. This is not true of the floors if they are reinforced concrete slabs poured around the steel beams and girders, and it is the belief of the writers that the effective moment of inertia of the floor girders, used in calculating the values of K , should include an estimated amount for stiffening effect of the slab. This might be done by considering the top flanges of the girders to be augmented by a part of the concrete slab as is done in calculating T-beams.

It has been suggested by several engineers, that the floor-slabs constitute a rigid diaphragm at each floor, causing all the columns to deflect equal amounts and that, therefore, the distribution of stresses should be calculated by considering the building as a whole instead of bent by bent. If it is possible to make the bents of stiffnesses which are symmetrical about the center line of the building, this method offers some advantages. If the bents are stiffer at one end of the building than at the other, due to the building being wider at one end than the other, or to a different column arrangement, the wind forces will produce a torsional effect as was clearly illustrated in the Meyer-Kiser Building in Miami, Fla.‡

Many rumors are heard as to the amount of deflection occurring in tall buildings during wind storms, but few authentic measurements are on record. A very small movement, if rapid, will cause uneasiness and apprehension on the part of tenants, while much larger movements with a period of vibration of 30 sec., or more, would be unnoticeable. The deflection and time of vibration of a 17-story building, 40 ft. wide, were measured during a storm in which the wind reached a velocity of 44 miles per hour, by Cyrus A. Melick, M. Am. Soc. C. E., in 1910.§ The deflection was from $\frac{3}{8}$ to $\frac{1}{2}$ in. and the period of vibration about 8 sec. The calculated deflections of this building indicate that the walls and partitions reduced the deflection about 40 per cent. Measurements of a like nature are now being made in the American Insurance Union Build-

* *Bulletin No. 80*, Univ. of Illinois Eng. Experiment Station, Urbana, Ill.

† *Journal*, Western Soc. of Engrs., Vol. 20 (1915), p. 341.

‡ Final Report of Committee of the Structural Division on Florida Hurricane, *Proceedings*, Am. Soc. C. E., August, 1928, Papers and Discussions, p. 1758.

§ *Bulletin No. 8*, Coll. of Eng., Ohio State Univ., Columbus, Ohio.

ing, at Columbus, Ohio, and it is hoped that they will reveal more facts about this interesting and important problem.

Reverting again to the assumptions made in calculating wind stresses, Mr. Orrell* calls attention to the effect of direct stress in the columns on the bending moments in the girders. The direct stresses in the columns, due to wind, are the results, and not the cause, of the bending stresses in the girders. The direct stresses in the columns are the algebraic sums of the shears in the connecting girders, and while they will probably modify the stresses slightly, their total effect will be small.

The curve (Fig. 8) which Mr. Ebling presents,† locating the points of contraflexure in the basement-story columns, might well be used instead of the analysis given in the paper.‡ In fact, the writers question the validity of the assumption that the bases of the basement-story columns are fixed by construction. Careful measurements of the dead load unit stresses around the periphery of ten of the basement-story columns of the American Insurance Union Building§ indicate dead load bending moments near the bases which are probably produced by base slabs that are not set absolutely level. The maximum bending moment developed would be accounted for by a variation in level of $\frac{1}{8}$ in. per ft. In this worst column the dead load unit stress on one side is about 10 000 lb. per sq. in.; while on the opposite side it is less than 2 000 lb. per sq. in. Inaccuracies in leveling the column bases or subsequent small unequal settlements would destroy the fixed end condition.

In conclusion, the writers wish to repeat the statement|| of Mr. Fleming:

"Let it be emphasized that no method should be used for the sole reason that it can be worked quickly and with ease. Safety should always come first, regardless of the time it takes to provide for it. Lack of time to make a better one is not a valid excuse for an engineer to offer for a poor design."

* *Proceedings, Am. Soc. C. E.*, January, 1929, Papers and Discussions, p. 199.

† *Loc. cit.*, October, 1928, Papers and Discussions, p. 2399.

‡ *Loc. cit.*, May, 1928, Papers and Discussions, p. 1416.

§ *Bulletin No. 40*, Eng. Experiment Station, Ohio State Univ.

|| *Proceedings, Am. Soc. C. E.*, October, 1928, Papers and Discussions, p. 2396.

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THE STIFFNESS OF SUSPENSION BRIDGES

Discussion*

By S. TIMOSHENKO, Esq.†

S. TIMOSHENKO,† Esq. (by letter).§—The advantages of applying trigonometric series to the investigation of cable stresses in suspension bridges were confirmed in several papers published during 1928. Martin|| proposed this method independently at about the same time as the writer, and used it in discussing the paper by Leon S. Moisseiff, M. Am. Soc. C. E., on the Delaware Bridge.¶ Professor Steuermann** showed that the method can be applied not only when the moment of inertia of the stiffening truss is constant within the entire span, but also when it is variable. F. Karpinski†† obtained very satisfactory results for the bending moments in the stiffening truss by using the writer's trigonometric series method for the deflection of the truss, together with the approximate formulas of J. Resal‡‡ and G. Pigeaud§§ for the calculation of the change in cable stress. Professor Priester||| developed the application of the trigonometric series method for cases in which the cable is attached to the towers and a concentrated load acts on the span. On the basis of this work, he concludes that the numerical calculations in the "trigonometric series method" are simpler than those in the "deflection method."

In discussing the paper, Mr. Frost¶¶ concludes that the practical value of the proposed method as a means of lessening the work is somewhat doubtful since it "is of little value to the busy engineer in the practice of his profession." The writer must apologize if the title of the paper seems misleading,

* Discussion of the paper by S. Timoshenko, Esq., continued from December, 1928, *Proceedings*.

† Author's closure.

‡ Prof. of Eng. Mechanics, Univ. of Michigan, Ann Arbor, Mich.

§ Received by the Secretary, March 28, 1929.

|| *Engineering*, Vol. 123 (1927), p. 506; see, also, *Engineering*, Vol. 125 (1928), p. 1.

¶ *Journal*, Franklin Inst., October, 1925.

** *Proceedings*, Am. Soc. C. E., December, 1928, Papers and Discussions, p. 2737.

†† *Annales des Travaux Publics de Belgique*, Juin, 1928.

‡‡ "Cours de Ponts Metalliques," Tome II, Premier fascicule, 1912.

§§ *Le Genie Civil*, 2 juillet, 1927.

||| Paper to be published as an *Engineering Research Bulletin*, Univ. of Michigan, Ann Arbor.

¶¶ *Proceedings*, Am. Soc. C. E., October, 1928, Papers and Discussions, p. 2405.

or suggests the advisability of applying his method to the preparation of cost estimates. The intention was to develop a method of analyzing cable stresses in suspension bridges without involving some of the arbitrary assumptions of the usual method, and, at the same time, introducing one that would be simple enough for practical application. The fact that a more rigorous solution is obtained with less work, justifies the application of series from the engineering point of view. The writer agrees with Mr. Frost that the paper is of the kind that serves only a small minority, and that it is of little value to the busy engineer. However, from time to time, it becomes important to discuss the fundamental assumptions upon which the usual methods of engineering analyses are based, to be sure that they are accurate enough and simple enough.

Professor Rode,* referring to the first numerical example of the paper,† raises the question as to the change in the cable stress due to the live load. This change can be calculated by the method developed in the paper. It can be shown in this case that it is small, and can be neglected in calculating the approximate value of the deflection. Referring to Equation (23),‡ it is regrettable that the complete derivation was not given, in order to compare calculations made on the basis of the usually accepted Equation (4)§ with those made from Professor Rode's more accurate equation.

The discussion|| of Professor Steuermann represents a very interesting development of the proposed Fourier Series method. It is shown by using this method that the case of a truss with variable rigidity can easily be analyzed. In this way Professor Steuermann arrives at the interesting conclusion that a considerable reduction in rigidity at the supports of a truss has only slight effect on the magnitude of the cable stress. This result is of practical importance, and suggests the advantage of designing stiffening trusses of variable rigidity.

The discussion¶ by Professor Priester represents several interesting examples of the application of the proposed method. It contains also calculations which prove the accuracy of this method in determining the cable stress. The discussion of the live load applied in two increments establishes the error which may be expected in applying the principle of superposition in discussing such structures as suspension bridges. The stress analysis with the cable fixed at the towers gives a simple solution in the form of a series to a problem which was formerly solved only by the tedious "cut and try" method. By using the series method, Professor Priester also obtained the influence lines for the cable stress and for the bending moments. These permit the determination of the most unfavorable loading for each cross-section of the truss.

In conclusion, the writer wishes to thank those who, by their discussion, contributed to the completeness and usefulness of his paper.

* *Proceedings, Am. Soc. C. E.*, October, 1928, Papers and Discussions, p. 2406.

† *Loc. cit.*, May, 1928, Papers and Discussions, p. 1467.

‡ *Loc. cit.*, October, 1928, Papers and Discussions, p. 2407.

§ *Loc. cit.*, May, 1928, Papers and Discussions, p. 1466.

|| *Loc. cit.*, December, 1928, Papers and Discussions, p. 2737.

¶ *Loc. cit.*, p. 2741.

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ENGINEERING FOUNDATION COMMITTEE ON ARCH DAM INVESTIGATION

ARCH DAM INVESTIGATION

Discussion*

By B. F. JAKOBSEN, M. AM. SOC. C. E.

B. F. JAKOBSEN,† M. AM. SOC. C. E. (by letter).‡—The main purpose of these tests is to correlate the test values with those obtained from existing formulas; or, in case this fails, to devise formulas which will agree with the tests. In Section 68,§ Professor Slater compares the observed and computed arch stresses. The writer was inclined to expect a disagreement due to the fact that the dam was badly cracked, which might prove a disturbing element, sufficient to vitiate the results of this immense amount of work. Fortunately, this is not so for the arch at Elevation 30. The theoretical formulas for determining stresses in arches are based on assumptions which innumerable tests have shown to be approximately correct. A serious discrepancy between theoretical and test values, therefore, could not be accepted as proof that the theory is incorrect.

Arch Stresses at Elevation 30.—The writer has chosen to investigate the stresses at Elevation 30 where the water is even with the crest of the dam, because the test is rather complete.¶ In order to compare test results and theoretical values, the stresses were computed for a number of points by the Cain formulas;¶ this implies that the arch load, p , is the same for every point of the arch, which is only approximately correct. Professor Slater has shown** that the formulas for foundation deformation developed by Dr. Vogt agree closely with test results. Dr. Vogt has shown that the yielding of the foundation can be approximately taken account of by assuming an

* Discussion on the Report of the Committee of Engineering Foundation on Arch Dam Investigation, continued from April, 1929, *Proceedings*.

† Cons. Engr. (La Rue & Jakobsen), Los Angeles, Calif.

‡ Received by the Secretary, February 8, 1929.

§ Report of Committee on Arch Dam Investigation, *Proceedings*, Am. Soc. C. E., May, 1928, Pt. 3, p. 187.

¶ *Loc. cit.*, Section 68, p. 187; see, also, Fig. 114, p. 163.

¶ *Transactions*, Am. Soc. C. E., Vol. LXXXV (1922), p. 233.

** *Proceedings*, Am. Soc. C. E., May, 1928, Pt. 3, Section 42, p. 124; also, Fig. 82, p. 128.

unyielding foundation at some distance, h_1 , from the actual foundation.* When the moment is of greatest importance in producing deformation, Dr. Vogt gives $h_1 = 0.45 t$, in which, t is the thickness of the arch. When the normal force and shear are the more important, h_1 must be given a larger value. In the arch under consideration, the influence of the moment is the smaller; since $t = 2$, $h_1 = 0.45 t = 0.9$ ft., which, as stated, should be augmented. Professor Westergaard, who takes account of the yielding of the foundation, finds $h_1 = 1.72$ for bending and $h_1 = 11.2$ ft. for the arch thrust.† In the following calculations, $h_1 = 1.25 \times 0.01745 \times 99 = 2.16$ ft. has been used as a conservative mean for bending and arch thrust. This corresponds to an increase of $1^\circ 15'$ for the half central angle. The constants of the

arch are‡ $t = 2$ ft.; $r = 99$ ft.; $\frac{t}{r} = 0.0202$; and the central angle, $2\phi_1 = 43^\circ 30' + 2^\circ 30' = 46^\circ$; so that $\phi_1 = 23$ degrees. Professor Cain's Equation (16)§ gives:

$$(p r - P_0) = \frac{C}{12} \frac{t^2}{r^2} p r = 0.0558 p r \dots \dots \dots (147)$$

The constant, C ($= 1640$), is found by interpolation from Professor Cain's Table 6.|| The arch thrust and the moment are as given by his Equations (4)¶ and (13),** respectively. The results of the calculations are given in Table 47.

TABLE 47.—LOAD STRESSES AT DOWN-STREAM FACE BY THE CAIN FORMULAS.

Values of ϕ , in degrees.	STRESS AT DOWN-STREAM FACE, IN POUNDS PER SQUARE FOOT.		
	Due to thrust.	Due to bending.	Total.
0	+46.7 p	-21.8 p	+24.9 p
4.5	+46.7 p	-19.3 p	+27.4 p
9.0	+46.8 p	-11.8 p	+35.0 p
13.5	+46.8 p	+ 0.7 p	+47.5 p
18.0	+46.9 p	+18.3 p	+65.2 p
21.75*	+46.9 p	+36.5 p	+83.4 p
23.0	+47.0 p	+43.5 p	+90.5 p

* At abutment.

The stresses due to temperature variations and shrinkage must also be considered. That they were present is shown by the various cracks. Professor Cain gives, for the thrust at the crown:††

$$H = C \frac{I}{r_2} E e t_0 \dots \dots \dots (148)$$

* Transactions, Am. Soc. C. E., Vol. 90 (1927), p. 555.

† Proceedings, Am. Soc. C. E., May, 1928, Pt. 3, p. 246.

‡ Loc. cit., Table 1, p. 15.

§ Transactions, Am. Soc. C. E., Vol. LXXXV (1922), p. 237.

|| Loc. cit., p. 266.

¶ Loc. cit., p. 236.

** Loc. cit., p. 237.

†† Loc. cit., p. 245.

in which, $I = \frac{8}{12}$, E is the modulus of elasticity, in pounds per square foot, e is the expansion per foot for a rise of 1° Fahr., t_0 is the change in temperature, and $C = 1\ 640$, as before. Let $k = E e t_0$, which is negative for a drop in temperature and for shrinkage. The arch thrust and the moment are then, as given by Professor Cain,* $H = 0.1115 k$ and $M_h = 0.115 k \left(\cos \phi - \frac{\sin \phi_1}{\phi_1} \right)$.

The stresses in the down-stream face (see Table 48) were found by the

$$\text{equation, } s = \frac{H}{t} + 6 \frac{M_h}{t^2}.$$

TABLE 48.—STRESSES IN DOWN-STREAM FACE DUE TO TEMPERATURE VARIATIONS, AS DETERMINED BY THE CAIN FORMULAS.

Values of ϕ , in degrees.	TEMPERATURE STRESSES IN DOWN-STREAM FACE, IN POUNDS PER SQUARE FOOT.		
	Due to thrust.	Due to bending.	Total.
0	+ 0.0558 k	+ 0.00446 k	0.0603 k
4.5	+ 0.0555 k	+ 0.00394 k	0.0594 k
9.0	+ 0.0550 k	+ 0.00340 k	0.0574 k
13.5	+ 0.0543 k	+ 0.00015 k	0.0541 k
18.0	+ 0.0530 k	— 0.00372 k	0.0493 k
21.75*	+ 0.0517 k	— 0.00745 k	0.0443 k
23.0	+ 0.0514 k	— 0.00885 k	0.0425 k

* The abutment is at $\phi = 21.75$ degrees.

The horizontal strains for reservoir full are shown in Fig. 114.† The vertical strains for sections at the center line, 10 ft., and 30 ft. off the center line, were scaled from Fig. 126‡ as —77, —80, and —65, or an average of about —75. (These include strains produced by vertical shrinkage.) The horizontal stress, σ_h , is found by Equation (29).§ When $E = 3\ 600\ 000$ and $\mu = 0.15$, the result, in pounds per square inch, is $\sigma_h = 3.68 \epsilon_h \left(1 - 0.15 \frac{75}{\epsilon_h} \right)$, in which, ϵ_h is the horizontal strain, as given in Fig. 114. The horizontal strains and the corresponding stresses are given in Table 49.

From the stresses given in Tables 47 to 49 the most probable values of p and k can now be determined by the method of least squares. The "errors", or deviations, are, $\epsilon_1 = (24.9 p + 0.0603 k - 234.5)$, $\epsilon_2 = (27.4 p + 0.0594 k - 246.0)$, etc. Forming $\epsilon_1^2 + \epsilon_2^2 + \epsilon_3^2 + \dots$, differentiating with respect to p and k , and equating these two formulas to zero, gives $p = 9.8005$ lb. per sq. in., or 1 411 lb. per sq. ft., and $k = -680.739$ lb. per sq. in. Table 50 gives the result and also the deviations or departures from the measured stresses.

* Transactions, Am. Soc. C. E., Vol. LXXXV (1922), p. 245.

† Proceedings, Am. Soc. C. E., May, 1928, Pt. 3, p. 163.

‡ Loc. cit., p. 174; see, also, Fig. 154, p. 229.

§ Loc. cit., p. 162.

TABLE 49.—HORIZONTAL STRAINS AND STRESSES AT DOWN-STREAM FACE FROM TEST.

Values of ϕ , in degrees.	ϵ_h , in millionths.	σ_h , in pounds per square inch.
0	75	234.5
4.5	78	246.0
9.0	91	293.5
13.5	118	393.0
18.0	160	547.5
21.75	240	842.0

The sum of the errors squared is 9 047, and the probable error of a single observation is, $0.675 \sqrt{\frac{9\ 047}{6-2}} = 32.1$ lb. per sq. in., so that $p = 9.8 \pm 0.50$ and $k = -681 \pm 480$. The average value of the measured stresses is $\frac{2\ 556.5}{6}$

$= 425$ and the probable error of a single observation is thus about 7.6% of the average value of the stresses measured. This must be considered an excellent check and a verification of the Cain formulas. The value of p found; that is, 1 411 lb. per sq. ft., is about 100 lb., or 7% less than the average value from the test. (It should be noted that p refers to the mean radius and should be decreased 1% when applied to the up-stream face.) The agreement between calculated and measured stresses could be made better if the foundation deformations were treated separately for arch thrust and for bending, or if h_1 was determined by the method of least squares; but the agreement is quite good enough without that. Only very rarely can the engineer hope to calculate the stresses with a probable error of less than 10 per cent. The value of $k = E e t_o$ was -680.739 . According to Professor Slater a value of $E = 3\ 600\ 000$ agrees best with the results* and according to Professor Davis,† e is about 0.000004, so that $E e = 14.4$ and $t_o = -47.3^\circ$ Fahr. According to tests made by Professor Davis, a specimen in air shrank 0.75 in. per 100 ft., in six months, or 625 per million,‡ which is equivalent to a cooling of about 160° Fahr. The value found for k is, therefore, plausible enough.

At the up-stream face at the abutment ($\phi = 21.75^\circ$), the load stress is, $p (46.9 - 36.5) = 101.9$ lb. per sq. in. compression (see Table 47). From Table 48 the tension due to shrinkage is $(0.0517 + 0.0075) k$, or 40.3 lb. per sq. in., so that the resulting stress is 61.6 lb. per sq. in. compression. The arch is, therefore, in compression throughout and the shrinkage cracks in this arch, if any, should have no great influence on the stress distribution inside the arch.

A comparison of the measured strains in the concrete and in the celluloid model at Elevation 30, shows§ both strains equal at the center line.|| Away from the center line, the concrete strains are the smaller, to a point about 30 ft. off the center line, at which both strains are again alike. Nearer the

* *Proceedings, Am. Soc. C. E.*, May, 1928, Pt. 3, p. 155.

† *Loc. cit.*, Table 29, p. 214.

‡ *Loc. cit.*, p. 205.

§ *Loc. cit.*, Fig. 153, p. 229.

|| *Loc. cit.*, Fig. 114, p. 163.

abutment than this, the celluloid strains are the greater. This indicates shrinkage stresses in the concrete (see Table 50), and too rigid an abutment in the model, which means a decrease of Dr. Vogt's h_v , so that the model in this respect does not represent actual conditions. The strains of equal intensity in the celluloid model are offset with respect to the center line, indicating that perhaps a slippage took place at one of the abutments; something happened which offset the strains. This comparison of the strains in the concrete and in the model also corroborates the Cain formulas.

TABLE 50.—COMPARISON OF STRESSES IN DOWN-STREAM FACE.

Values of ϕ , in degrees.	STRESSES IN DOWN-STREAM FACE, IN POUNDS PER SQUARE INCH, BY THE CAIN FORMULAS.				
	Load.	Temperature.	Total.	By tests.	Error.
0	244.03	- 41.05	202.98	234.5	- 31.52
4.5	268.53	- 40.44	228.09	246.0	- 17.91
9.0	318.02	- 39.07	303.95	298.5	+ 10.45
13.5	405.62	- 36.88	428.09	393.0	+ 35.69
18.0	638.99	- 33.56	605.43	547.5	+ 57.93
21.75	817.36	- 30.09	787.27	842.0	- 54.73
.....	2 556.5	+ 0.09

It is of interest to determine p on the assumption that $k = 0$. Professor Slater made an attempt to exclude shrinkage;* if there was no crack formed, then there was no shrinkage to consider and $k = 0$ would be the correct assumption. If there was a crack, as, for example, a slight lifting away from the abutments at no load, then $k \neq 0$ is the correct assumption. Assuming $k = 0$, gives $p = 9.1813$ lb. per sq. in. by the method of least squares. The sum of the errors squared is 11 131.7 and the probable error of a single measurement is,

$$0.675 \sqrt{\frac{11\,131.7}{5}} = 31.4 \text{ lb. per sq. in.}$$

and,

$$p = 9.18 \pm 0.25$$

The "fit" is as good as before when shrinkage was included. From this evidence alone it would be impossible to decide whether or not there was shrinkage to be included, but the comparison between the strains in the test dam and those in the celluloid model, previously mentioned, leads the writer to conclude that shrinkage was present.

When $k = 0$, $p = 9.1813$ and when $k \neq 0$, $p = 9.8005$; the difference between these two values of p is due to the transfer of load from the arches to the cantilevers, when shrinkage occurs. Due to the fact that the shrinkage reduces the stresses (see Table 50), p must be greater when shrinkage is included in order to give the same stresses.

* Proceedings, Am. Soc. C. E., May, 1928, Pt. 3, pp. 117 and 118.

The transfer of load from the arches to the cantilevers, or *vice versa*, which accompanies temperature variations (or their equivalents, such as swelling and shrinkage), is a phenomenon that has been repeatedly stated by Professor Cain* and the writer has called attention to this transfer of load due to swelling of the up-stream face.† Mr. Noetzli on the other hand continues to disregard this transfer of load and has done so in the design data presented. The test data support Professor Cain's contention, as was to be expected, because if shrinkage deforms the arch and causes a displacement, it must affect the division of load between the arches and the cantilevers.

The probable error of a single determination when k was assumed to be different from zero, was found previously as 32.1 lb. per sq. in. and when k was assumed equal to zero the probable error of a single determination was 31.4 lb. per sq. in. Calculating the probable errors of p and k , these values, with their probable errors,‡ are: $p = 9.80 \pm 0.50$ and $k = -681 \pm 480$, and when $k = 0$, $p = 9.18 \pm 0.25$.

An inspection of Table 50 shows the stress in the down-stream face of the arch to vary continuously from 234.5 at the crown to 842.0 at the abutment. If the cylinder formula was valid, the stress should remain constant and equal to the mean of the six values of s_h in Table 50, or 426.08, and the probable error of a single observation would then be 159 lb. per sq. in., or about five times as great as when the Cain formulas are used.

Shear at the Foundation for Cantilever 30 Feet Off Center Line and Arch at Elevation 20.—Let p be the water pressure on a surface element close to the abutment; let c be that part of p which is transferred vertically downward through the cantilever; and $p - c$, consequently, the part transferred through the arch, horizontally. Let θ be the angle between the horizontal and the abutment, then,§

$$\frac{p - c}{\sin \theta} = \frac{c}{\cos \theta} \dots \dots \dots (149)$$

This follows, because the load, p , applied very close to the abutment has no influence on the stress distribution away from the abutment. Referring to Fig. 60,|| it will be seen that a point at Elevation 20 and 30 ft. off the center line, lies on the abutment. Referring to Fig. 132,¶ to the section 30 ft. off the center line, it may be seen that if the arch and cantilever load lines are prolonged down to Elevation 20 from Elevation 25, the two loads are equal, as they should be, because $\theta = 45^\circ$ in the test dam. Each load is, therefore, 1 250 lb. per sq. ft.

Equation (149) holds also for the total shear transmitted by an arch and by the corresponding cantilever, which ends in the same point on the abutment

* Transactions, Am. Soc. C. E., Vol. LXXXIV, (1921), p. 80, and Vol. 90 (1927), p. 544.

† Loc. cit., Vol. 90 (1927), p. 517.

‡ "The Theory of Errors and Method of Least Squares," by William W. Johnson, 1920 Edition, Chapter VIII, N. Y., John Wiley & Sons.

§ Proceedings, Am. Soc. C. E., April, 1928, Papers and Discussions, Equation (51), p. 1312.

|| Loc. cit., May, 1928, Pt. 3, p. 107.

¶ Loc. cit., p. 182.

as the arch and this affords an independent check on the test results. In Fig. 132, for the section 30 ft. off the center line, the loads on the cantilever are zero from the top to Elevation 40 and from there down they are, as follows:

Elevation.	Cantilever Loads, in pounds per square foot.
35	200
30	300
25	650
20	1 250

The total cantilever shear is, therefore, $5 (100 + 300 + 650 + 625) = 8\,375$ lb.

The arch loads at Elevation 20, as obtained from Fig. 132, are:

Section.	Load, in pounds per square foot.
At center	1 300
10 ft. from center	1 450
30 ft. from center (at abutment)	1 250
Approximate mean	1 350

When shrinkage stresses are neglected,* the arch shear as given by Professor Cain is $S = (pr - P_0) \sin \phi$. The dimensions of the arch are,† $r = 98.73$ ft., t (as built) = 2.58 ft., and $2\phi_1 = 31^\circ 30' + 2^\circ 30' = 34^\circ$, or $\phi_1 = 17$ degrees.

The value of C in Equation (147) was found by calculation to be 4367; $(pr - P_0) = 0.2472 pr$; and the arch shear at the abutment, $\phi = 15.75^\circ$, is $S = 0.2472 \times 1\,350 \times 98.7 \times 0.2714 = 8\,950$ lb. This is a very good check of the cantilever shear, which was found to be 8375 lb. It is probably not correct, because shrinkage has not been included in the calculations, as it should have been. If a value of $k = -680.7 \times 144$ lb. per sq. ft. is used, the same as was found for the arch at Elevation 30, Equation (148) gives $H = 62\,800$ lb. and the total shear is, $S_s = (pr - P_0 + H) \sin \phi_a = 26\,000$ lb. This is much larger than the cantilever shear. If the cantilever carried all the load, the shear would be $31.25 \times 1\,600 = 50\,000$ lb.

The difficulty with this arch is, that there is some tension in the down-stream face, near the crown,§ and, therefore, considerable tension in the arch in the up-stream face of the abutment. If this tension produces a crack, the Cain formulas do not apply|| and the comparison between the arch strains in the test dam and in the celluloid model indicate cracking, as do also the calculations previously given. (There is no tension in the down-stream face of the celluloid model at Elevation 20. The least stress is that corresponding to a compressive strain of 25). Tests to show the influence of cracks on the distribution of the stresses would be helpful to the engineer in designing dams.

* Transactions, Am. Soc. C. E., Vol. LXXXV (1922), p. 238.

† Proceedings, Am. Soc. C. E., May, 1928, Pt. 3, Table 1, p. 15.

‡ Loc. cit., Fig. 2, p. 16.

§ Loc. cit., Fig. 153, p. 229.

|| Transactions, Am. Soc. C. E., Vol. LXXXV (1922), p. 272.

Professor Slater's reductions, which fail to take account of shrinkage cracks, are subject to error on that account. The influence of shrinkage would be to decrease the load on the arches and increase that of the cantilevers, and the preceding calculations also indicate that. The influence of vertical shrinkage is to decrease the measured tension and increase the measured compression and thus indicate a larger compression in the vertical direction than actually exists. It may perhaps be possible to go over the reductions and attempt to take account of shrinkage and cracks. It would also be interesting and helpful to have some probable errors determined and the theory of probability could no doubt be made good use of to increase the reliability of the test results.

Comparison of Arch Loads as Found by Design and by Test.—Table 51 shows the arch loads, as found by designing for equal deflections and no change in temperature (see Table 1*). The loads, as determined by Professor Slater from the test, are taken from Fig. 132, and the abutment loads are added; these were determined by Equation (149). The writer would have used Table 4,† which includes a temperature drop of 20° Fahr., except for the fact that it shows greater arch loads, while the writer thinks the reverse should hold.

For a temperature drop of 20° Fahr. Table 6‡ shows a tension of 292 lb. per sq. in. at the extrados of the abutment at Elevation 30, while the writer's calculations indicate compression although the temperature drop is about 47° Fahr., or the shrinkage is equivalent to that. Mr. Noetzli used a temperature coefficient of 0.0000055 which is the customary value, while the writer used the coefficient found by Professor Davis. When correcting for this, the two values agree quite closely. The writer, however, finds a compression of 61.6 lb. per sq. in., as stated on page 1244. In Fig. 114, the bending strain at the abutment equals about 100 and the total strain about 240, so that the strain due to the arch thrust is about 140. If the bending stress is linear, as no doubt it is, at least approximately, the strain at the extrados at the abutment would be 40 in compression or about one-half the strain found at the center line of the down-stream face (see Fig. 114). At this point the writer found a compression of 203 lb. per sq. in. and one-half of this checks the 62 lb. per sq. in. fairly well. It might check better if Poisson's ratio was introduced, but this may also change the bending stresses somewhat. At any rate it is clear that the compression of 62 lb. per sq. in. found by the writer is much closer to the truth than the tension of 292 lb. per sq. in. used in the design.

The load distribution used in the design, assuming no temperature change, involves cantilever tension up to 380 lb. per sq. in., and if a drop of 20° Fahr. is included, the maximum tension is 560 lb. per sq. in. (see Table 6‡). This load distribution can be significant of nothing, in particular, since concrete will not stand such high tension. Table 3§ gives a vertical tension of 307 lb.

* *Proceedings, Am. Soc. C. E.*, May, 1928, Pt. 3, p. 15.

† *Loc. cit.*, p. 19.

‡ *Loc. cit.*, p. 20.

§ *Loc. cit.*, p. 17.

per sq. in. in the down-stream face at Elevation 30, while Fig. 154* for the test dam shows a strain of 50 corresponding roughly to $\frac{2}{3} \times 200 = 133$ lb. per sq. in., or less than one-half. For the celluloid model, the strain is 125, or 2.5 times greater, say, about 332 lb. per sq. in., which checks Mr. Noetzli's value fairly well. The reversal from tension to compression in the down-stream face of the central cantilever occurs in Table 3 between Elevations 30 and 20, but nearer Elevation 20, and this also agrees closely with the celluloid model, while in the test dam the transition occurs a little lower down.

TABLE 51.—LOADS ON ARCHES, IN POUNDS PER SQUARE FEET.*

Elevation.	Water pressure on up-stream face.	From design data at crown.	LOADS CARRIED BY ARCHES.				
			From Test Data.				
			Crown.	10 ft. off.	30 ft. off.	Abutment.	Distance from crown.
50	625	775	950	800	750	312	60
40	1 250	1 210	1 300	1 500	1 400	625	50
30	1 875	1 335	1 500	1 500	1 700	937	40
20	2 500	1 250	1 300	1 450	1 250	1 250	30
10	3 125	875	1 300	1 300	1 562	20

* Water at Elevation 60.

† No temperature change.

It is important to emphasize this disagreement between calculated and measured cantilever stresses and the corresponding loads, since it is this fact that has prompted some engineers† to insist that the arches be calculated for full hydrostatic pressure, and then perhaps allow a somewhat higher unit stress to account for the unknown amount of load which the cantilever may carry. Referring to Table 7,‡ it will be seen that, at Elevation 50, the design stress of 775 lb. per sq. in. is higher than the full hydrostatic head, which is 625 lb. per sq. in., and the average load, as determined by Professor Slater from the tests, is between the two. A properly designed dam has always a considerable margin of safety near the top, however, because it is impossible to know the exact maximum head that may exist, and an increase in head is important near the top. At Elevation 10, Table 7 shows a design stress of only 875 lb. per sq. in., while the average is probably about 1 350 lb. per sq. in., according to the tests. As noted previously, these values may be somewhat too high due to the fact that shrinkage stresses were not included in the calculations.

The writer would like to record a few inconsistencies that occur in Table 9.§ For instance, it appears desirable and just that the notation in the

* *Proceedings*, Am. Soc. C. E., May, 1928, Pt. 3, p. 229.

† *Transactions*, Am. Soc. C. E., Vol. 90 (1927), pp. 520-553; Vol. LXXXV (1922), p. 253.

‡ *Proceedings*, Am. Soc. C. E., May, 1928, Pt. 3, p. 22.

§ *Loc. cit.*, p. 24, et seq.

twentieth column opposite Dam No. 19* should read "Variable Radius, Constant Angle Arch Type", to distinguish it from Dam No. 50.† To the "List of Literature" on this dam should be added, "*Engineering and Contracting*, June 8, 1920".

Furthermore, it should be noted that the writer was the designer of the Pacoima and Big Santa Anita Dams (Nos. 43‡ and 44‡) as they were actually constructed, and this should have been so acknowledged in Table 9. To the literature on those two dams should be added, "*Transactions, Am. Soc. C. E.*, Vol. 90 (1927) p. 475".

When the proposed tests on models, now in progress§ under the supervision of J. L. Savage and Ivan E. Houk, Members, Am. Soc. C. E., have, in a measure, been completed, it is to be hoped that a new large scale test can be undertaken. Such a dam should be properly grouted and have a much larger central angle than the Stevenson Creek Dam so as to represent, more nearly, a real arch dam.

It is clear that Professor Beggs' tests|| can be of considerable value, when their limitations are kept in mind—that is, the difficulty in reproducing the conditions at the abutments correctly. Celluloid does not crack under tension corresponding to that which produces cracks in concrete and, furthermore, shrinkage stresses occur in concrete. After all, what the designer needs is a reasonably correct estimate of the greatest stresses rather than information as to the exact distribution of the stress, which certainly is very complicated. The test has given conclusive evidence as to the correctness of the Cain formulas, as stated on page 1244, but little information has been obtained with respect to cantilever stresses in general and the writer feels that considerable uncertainty exists regarding the actual distribution of the water load as between the arches and the cantilevers.

The method of analysis used by Professor Westergaard,¶ treating the arch as a thin shell, is most interesting and will probably give a more nearly correct stress distribution than any other method. George Paaswell, M. Am. Soc. C. E., recently suggested a similar procedure.** The mathematical difficulties are certainly considerable and will be even greater for thick dams; but when dams costing \$25 000 000 and more, such as the proposed San Gabriel Dam of the Los Angeles County Flood Control, and the Boulder Canyon Dam, are involved, additional labor should mean but little, if better results can be achieved thereby. It would have been more satisfactory if Professor Westergaard had calculated the stresses found by his method, so that they could be compared with the test results.

The tests made by Professor Davis indicate that the plastic flow of concrete increases in direct proportion with the stress,†† so that it may be taken

* *Proceedings*, Am. Soc. C. E., May, 1928, Pt. 3, p. 27.

† *Loc. cit.*, p. 31.

‡ *Loc. cit.*, p. 28.

§ *Loc. cit.*, p. 215.

|| *Loc. cit.*, p. 219.

¶ *Loc. cit.*, p. 231.

** *Transactions*, Am. Soc. C. E., Vol. 90 (1927), p. 567.

†† *Proceedings*, Am. Soc. C. E., May, 1928, Pt. 3, Table 28, p. 213, and Fig. 144, p. 214.

account of approximately by a decrease of the modulus of elasticity. This means that the plastic flow will have little or no influence on the stress distribution, and that is fortunate.

The difficulties to overcome in this test have been great; they include temperature changes, shrinkage and swelling of the concrete, cracking of the dam, yielding of the foundations, changes in the cement, the variation of the modulus of elasticity, and time effect. Bearing this in mind, and the immense amount of painstaking labor necessary to obtain concordant results, the Committee and especially those who have done the actual work, are to be congratulated.

STREET DESIGNING FOR VARIOUS USES

Discussion*

By GEORGE W. THAYER, M. Am. Soc. C. E.

GEORGE W. THAYER, M. Am. Soc. C. E. (the writer) The writer is glad that his paper brought forth so much discussion and from so many different localities. He agrees fully with Colonel Ordelt that "location" is an important factor in street design. He holds, however, that it can be varied more with roads than with streets because most cities have greater freedom in the latter case.

Mr. Sawyer's comment on grades and cross sections may be considered as an attempt to provide satisfactory drainage and safety for travel of traffic, with thought as to the effect, both physically and financially, on abutting property. It is often a question of whether or not the pedestrian grade should be established at once or left until the public and jolting streets are the cause for a change.

The writer knows of one case in Chicago, N. Y., in which an original cut of 50 ft. was made, while in another instance, the street was torn up, well built up, and a cable railway installed below the pedestrian grade was established. In the Elkhart, at Omaha, the writer adopted a 15 ft. grade as a compromise on tracks over streets because the cars about the street for cars driven by horses.

The writer was much interested in Mr. Ordelt's discussion. It is worth being to know that city planning should be made at least as early as in ancient times. Probably the Egyptian pyramids were planned in the places where buildings were low and traffic by hand, such consideration is easily explained. Perhaps, however, the pyramids were not made upon these considerations.

* Discussion of the paper by George William Thayer, M. Am. Soc. C. E., presented at the Annual Meeting of the American Society of Civil Engineers, 1912, at New York.

1. Author's closure.

2. Civil Eng. in Design, 1911.

3. Received by the Department, March 21, 1912.

4. Proceedings, etc., Am. Soc. C. E., November, 1912, Paper and Discussion, p. 1241.

5. Ibid. p. 1241.

6. Ibid. p. 1241, March, 1912, Paper and Discussion, p. 1241.

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STREET DESIGNING FOR VARIOUS USES

Discussion*

BY GEORGE W. TILLSON, M. AM. SOC. C. E.†

GEORGE W. TILLSON,‡ M. AM. SOC. C. E. (by letter).§—The writer is very glad that his paper brought forth so much discussion and from so many different localities. He agrees fully with Colonel Crosby|| that "location" is an important factor in street design. He thinks, however, that it can be varied more with roads than with streets because land values have greater weight in the latter case.

Mr. Sawyer's comments¶ on grades are good. Grades must be established so as to provide satisfactory drainage and satisfy the needs of traffic, with thought as to the effect, both physically and financially, on abutting property. It is often a question of whether or not the permanent grade should be established at once or left until the public and property owners see the necessity for a change.

The writer knows of one case in Omaha, Nebr., in which an original cut of 60 ft. was made, while, in another instance, the street had been paved, well built up, and a cable railway installed before the permanent grade was established. In the Eighties, at Omaha, the writer adopted a 4% grade as a maximum on trolley car streets because that was about the limit for cars drawn by horses.

The writer was much interested in Mr. Simham's discussion.** It is gratifying to know that city planning received so much attention in India in ancient times. Probably few American engineers were aware of it. In places where buildings were low and traffic so light, such consideration is really surprising. Perhaps, however, elephants required more space than automobiles.

* Discussion of the paper by George William Tillson, M. Am. Soc. C. E., continued from March, 1929, *Proceedings*.

† Author's closure.

‡ Cons. Engr., La Grange, Ill.

§ Received by the Secretary, March 20, 1929.

|| *Proceedings*, Am. Soc. C. E., November, 1928, Papers and Discussions, p. 2613.

¶ *Loc. cit.*, p. 2615.

** *Loc. cit.*, March, 1929, Papers and Discussions, p. 815.

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PAPERS AND DISCUSSIONS

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mum on trolley car streets because that was about the limit for cars drawn
by horses.

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fying to know that city planning received so much attention in India in
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* Discussion of the paper by George William Tilson, M. Am. Soc. C. E., continued from
March, 1925, Proceedings.

† Author's closure.

‡ Cons. Grant, La Grange, Ill.

§ Received by the Secretary, March 20, 1925.

|| Proceedings, Am. Soc. C. E., November, 1925, Papers and Discussions, p. 2512.

¶ Loc. cit., p. 2512.

** Loc. cit., March, 1925, Papers and Discussions, p. 2512.

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BASE MAPS FOR REGIONAL PLANNING

Discussion*

By HARLAND BARTHOLOMEW, M. Am. Soc. C. E.†

HARLAND BARTHOLOMEW,‡ M. Am. Soc. C. E. (by letter).§—In closing the discussion, it is interesting to note that there is apparent unanimity of agreement in the value of the 2 000-ft. scale as the most important base map. Mr. Black|| and Mr. Lewis¶ mention the fact that this has been used for the principal studies in the Philadelphia and New York regions, respectively. Mr. Lewis makes a very interesting suggestion when he states** that the ideal base map for regional planning has not yet been drawn, and then gives specifications for such a map with which the writer is in entire accord.

While there has been little discussion of other maps which might be used as bases for regional planning studies, Mr. Lewis believes that scales of $\frac{1}{2}$ mile to the inch and 1 mile to the inch would be much more satisfactory than that of 2 miles to the inch, suggested in the paper. Actually, there has been insufficient work done in regional planning to offer much real experience of value on this point. Different conditions will necessitate the use of varied map scales. It will be a matter relating entirely to the character of the material to be displayed. In the paper it was assumed that this scale would be used primarily for the graphic portrayal of important factual material in skeleton form. Undoubtedly, in some instances, the scale of 1 mile to the inch, or of even $\frac{1}{2}$ mile to the inch, will lend itself better to work of this character.

Colonel Birdseye's criticism** is well taken, in that a 20-ft. contour interval as a standard would be most unfortunate, and, in fact, useless in certain areas.

* Discussion of the paper by Harland Bartholomew, M. Am. Soc. C. E., continued from January, 1929, *Proceedings*.

† Author's closure.

‡ City Plan and Landscape Engr. (Harland Bartholomew and Associates), St. Louis, Mo.

§ Received by the Secretary, March 25, 1929.

|| *Proceedings*, Am. Soc. C. E., January, 1929, Papers and Discussions, p. 234.

¶ *Loc. cit.*, p. 229.

** *Loc. cit.*, p. 233.

The contour interval will have to be determined entirely by local conditions. The 20-ft. interval would probably become standard in the great majority of instances, but it certainly should not be set up as an inflexible specification.

Mr. Sherman* cites an instance of the value of an air map. Because of the incomplete mapping in certain regions, an air map will usually prove to be of great value in regional planning studies. Until an accurate topographic map has been prepared, the air map will be most useful for many preliminary studies.

* *Proceedings, Am. Soc. C. E., January, 1929, Papers and Discussions, p. 234.*

BASE MAPS FOR REGIONAL PLANNING

Discussion*

By HAROLD BARTHOLOMEW, M. Am. Soc. C. E.

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While there has been little discussion of other maps which might be used for regional planning studies, Mr. Lewis believes that scales of 1 mile to the inch and 1 mile to the inch would be much more satisfactory than the 2 miles to the inch suggested in the paper. Actually, there has been important work done in regional planning to offer much real experience of value on this point. Different conditions will necessitate the use of varied map scales. It will be a matter relating entirely to the character of the material to be displayed. In the paper it was assumed that this scale would be used primarily for the graphic portrayal of important factual material in skeleton form. Undoubtedly, in some instances, the scale of 1 mile to the inch, or of even 1 mile to the inch, will lend itself better to work of this character.

Colonel Birdsey's criticism** is well taken, in that a 20-ft. contour interval as a standard would be most unfortunate, and, in fact, useless in certain areas.

* Discussion of the paper by Harold Bartholomew, M. Am. Soc. C. E., continued from January, 1929, *Proceedings*.

** Author's closure.

† City Plan and Landscape Eng. (Harold Bartholomew and Associates), St. Louis.

‡ Received by the Secretary, March 22, 1929.

§ *Proceedings, Am. Soc. C. E., January, 1929, Papers and Discussions, p. 234.*

|| See also p. 229.

** See also p. 232.

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STANDARD CITY ENGINEERING SURVEYS

Discussion*

By MESSRS. G. D. WHITMORE AND ROBERT H. RANDALL.†

G. D. WHITMORE,‡ Assoc. M. Am. Soc. C. E.—The projection used for the topographic maps on the 200-ft. scale which are described in the paper, is designed to fulfill the following requirements: (1) The map edges should be always true in azimuth (within the scale of the map), so that geographic bearings or azimuths can always be scaled directly without the necessity of computing the amount of convergence of meridians which is required on a plane or rectilinear projection; (2) the method of projection and co-ordination should provide for exact matching and co-ordinating of the city survey maps with maps prepared by other cities and the county, State, Federal Government, War Department, etc.—the usual basis of co-ordination on such maps being geographic positions based on North American Datum; and (3) the projection should provide a rectangular co-ordinate basis for ease and simplicity of ordinary distance, bearing, and position computations, and for convenience in plotting.

Engineers in general subscribe to the principle that all comprehensive maps showing elevations should be constructed on one common level datum, namely, mean sea level as defined by the bench-marks of the Federal Government. It is likewise desirable, in the speaker's opinion, that all such maps be constructed on one common horizontal co-ordinate system; and here, again, the only practical and unified co-ordinate basis is that defined by the geographic positions of the triangulation and traverse monuments of the Federal Government, known as North American Datum.

As a definite example of the utility of this method of projection, consider the case of two cities situated just a few miles apart, each making topographic surveys as a part of the comprehensive city surveys of their areas, the resulting

* Discussion of the paper by Robert H. Randall, M. Am. Soc. C. E., continued from January, 1929, *Proceedings*.

† Authors' closure.

‡ Vice-Pres. and Asst. Chf. Engr., R. H. Randall & Co., Inc., Toledo, Ohio.

maps being published on the same scale (1 in. equals 200 ft.). Each city adopts an originating point for its sheet layout which is an intersection of an even-minute meridian of longitude and an even-minute parallel of latitude, this point being defined and determined by triangulation or traverse connections to the survey monuments of the Federal or State Government. Both cities adopt a map sheet which is 60" wide in longitude and 40" high in latitude. When the map sheets of the two cities are extended so that they meet at some point between them, the sheet boundaries of the maps of the first city will coincide and match exactly with those of the second city. This agreement in position will also apply to all information shown at the joining edges wherever sheets from the two surveys meet. The rectangular co-ordinate lines of the two different sets of maps, however, will not coincide and will not be parallel.

The methods of computation and plotting of this projection are not difficult. An origin of rectangular co-ordinates which is an even-minute intersection of a parallel of latitude and a meridian of longitude is adopted near the center of the mapping area, through which the reference or true meridian of the plane co-ordinate system is passed. This point also serves as the beginning point of the sheet layout, from which the geographic co-ordinates of the sheet corners are readily computed. The origin is assigned a large rectangular co-ordinate value, for example, 100 000 ft. north and 100 000 ft. east. The geographic co-ordinates of the sheet corners are then converted to rectangular co-ordinates, referred to the central origin and reference meridian. From these co-ordinates, the ground size, in feet, of each map sheet is computed. The resulting figure is, within the scale of the map, a symmetrical trapezoid, narrower across the top than across the bottom. For sheets plotted on the 200-ft. scale this difference in width between top and bottom is equal to something less than a foot by ground measure (0.005 in., paper measure), so that it is hardly measurable, and for all practical plotting purposes the figures may be considered a simple rectangle. The boundaries of the sheet are plotted as a rectangle, of the size indicated, and they represent true geographic meridians and parallels on the surface of the earth. The rectangular co-ordinate lines are overlaid on this by plotting the intersections of the even 1 000-ft. rectangular lines with the geographic sheet boundaries. The distances of these intersections from the sheet corners are directly derivable from the rectangular co-ordinates of the corners. All subsequent plotting, such as triangulation and traverse control stations, etc., is accomplished by means of rectangular co-ordinates.

It is evident that all the advantages of the rectilinear method of projection are incorporated in this combined method. All local use of the map may be on a system of rectangular co-ordinates, just as if the map were plotted with sheet boundaries on rectilinear instead of geographic meridians and parallels. Both the geographic and rectilinear systems are shown, and it is optional with the user which one is chosen for local studies and uses.

In any discussion of the question of map construction by ground methods as against combined aerial and ground methods, the fullest consideration

must be given to scale and accuracy. The scale recommended by the author for topographic maps of city areas is 1 in. equals 200 ft. That scale has been adopted by about twenty-five or thirty cities in the United States, and those cities are generally considered to be the best equipped as far as basic engineering maps are concerned. That it will become practically a standard scale for this class of maps may be assumed.

A scale of 200 ft. to the inch is capable of indicating distances and positions within 1 ft., and the stadia readings and subsequent plottings on the map are, or should be, accurate within that amount. The field methods used in constructing these maps include an average of one instrument set-up for every 1 to 2 acres, with from five to thirty stadia readings per acre, depending on the type of country and amount of cultural detail. Thus, the topographer is seldom more than 100 or 200 ft. distant from the features being sketched, and large blunders are literally impossible. Elevations interpolated from such maps have proved to be, with rare exceptions, correct within less than one-half the contour interval. Profiles constructed from them are practically as accurate as those executed by transit and level.

In a few instances in the past advocates of the use of aerial photographs in conjunction with ground surveys for topographic map construction have cited cases in which the combined air and ground maps have disclosed large errors existing in previous maps made by ground methods. It is a practical certainty that such ground maps were not constructed with the same degree of accuracy and detail as those herein described. On the other hand, the ground-method maps, on the 200-ft. scale and constructed as noted, show much cultural and topographic detail not discernible in air photographs.

Another very important consideration is that of cost. All such comparisons should indicate the cost of the combined air and ground map, as against the cost of the ground map. The speaker is not aware of any combined-method maps of city areas, on the 200-ft. scale, equal in accuracy and amount of detail to the maps described by him, for which cost figures have been published.

ROBERT H. RANDALL,* M. AM. Soc. C. E. (by letter).†—In the discussion of the proper frequency of triangulation stations for controlling the city survey, Mr. Heaton‡ inclines to the view that the recommended distribution of one station for every 1 to 3 sq. miles would result in greater frequency than is necessary in the average city. It seems to the writer that this question is hardly capable of answer except in rather general terms. Considerable experience in executing traverse of high accuracy and in metropolitan areas leads the writer to believe that the distribution mentioned in the paper is likely to be the best investment.

The frequency of triangulation stations, of course, should always be considered in relation to the accuracy of the traverses which are to expand and make usable the triangulation. For the traverse, Mr. Heaton recommends an

* Pres. and Chf. Engr., R. H. Randall & Co., Inc., Toledo, Ohio.

† Received by the Secretary, March 25, 1929.

‡ *Proceedings*, Am. Soc. C. E., January, 1929, Papers and Discussions, p. 237.

average accuracy of 1 part in 50 000. As a matter of fact most of the first-order traverses executed under the specifications implied by the writer have a total average accuracy greater than that. The figure of 1 part in 30 000 given in the paper is meant to be a rather low limit. Within the down-town section of the City of Pittsburgh, Pa., for instance, traverses average 1 part in 90 000, ranging from 1 part in 150 000 to 1 part in 20 000 for an occasional small circuit.

In executing traverse under city conditions, the writer believes that the practical way to secure proper accuracy is to run traverses in relatively small circuits and connect with triangulation at approximately the intervals specified. It is believed that to attain the accuracy specified by Mr. Heaton (and which, as a matter of fact, is ordinarily attained) with triangulation connections at intervals of 4 to 9 miles, would require angular measurement of an accuracy which could only be attained by night work. It is not considered practical to expect the city's engineering forces to undertake night work, month after month, for the length of time necessary to accomplish the work in this manner.

Mr. Heaton states* that,

"On the completion of a geodetic control survey the city should then endeavor to make a cadastral survey connecting each individual piece of property to it, by determining the co-ordinates of the two corners which are on the property frontage".

The writer believes the first duty of the City is to locate and determine co-ordinate positions for all critical points upon the street lines. It is more important that the City be prepared to give the line of every street than that the boundaries between private properties should be settled. Under the present great and increasing use of the police power in many phases of city planning, the City, for the general public good, controls the use and development of private properties in many ways. Since city planning consists, in the last analysis, of the establishment of boundaries of streets and of districts to be devoted to various uses, it is only fair that the street lines to which all such boundaries are referred should be known beyond question, and that they should be readily available to the individual property owners whom they affect. After this is done, the matter of determining private boundaries may be undertaken. This will tend to be increasingly easy, through the stabilizing effect of the definite establishment of the street lines and the operation of statute limitations concerning adverse possession.

The discussion of Mr. Brown† regarding projection is largely answered in Mr. Whitmore's comment.‡ In regard to the use of air methods in the topographic mapping incidental to the city survey, the writer subscribes to the general expectation that there is a probability that some photographic and optical procedure will be developed by which maps of proper accuracy of the scale (1 in. = 200 ft.) recommended, may eventually be made by air methods. At present, however, it is not believed that the necessary controlling ground work plus the aerial processes result in a map of the specified accuracy at as

* *Proceedings, Am. Soc. C. E., January, 1929, Papers and Discussions, p. 238.*

† *Loc. cit., p. 239.*

‡ See p. 1257 *et seq.*

low a cost as the described ground methods alone. In general, it may be stated that aerial methods have received much publicity, as from the nature of the case—every one is increasingly interested in air matters—is natural. It is probable that considerable improvement can be and will be made in ground methods, but of this, engineers hear very little. Surveying is perhaps the oldest applied science. Improvement in method has generally kept pace with the increase in accuracy of results demanded. Methods are only to be valued in the light of their practical utility. Thus far, the utility of air methods as applied to surveys of the scale required for city topographic mapping has not, to the writer's belief, been demonstrated.

Mr. Pendleton's exposition* of aerial mapping methods is of interest. The writer's purpose in preparing this paper, however, was to give an account of present methods, in conformity with current practice in the majority of city surveys. At present, he is inclined to agree with Professor Finch† that for maps showing relief "in the majority of cases, ground methods still appear to offer the best economy."

Mr. Vedder‡ suggests a very practical means of financing the triangulation and traverse necessary to the city survey through funds raised for assessment maps. The cost of such control is a logical charge against assessment maps, and it may be pertinent to state, also, that the entire city survey is a reasonable and logical charge against sewer, highway, and water improvement projects.

As to the procedure of doing topographic mapping during winter weather in Northern cities, this is a question of policy which best may be settled by the particular conditions applying in any particular city. As a general thing, the writer does not believe this to be economical procedure. It seems probable that more mapping may be done for a given expenditure by treating the mapping program as a project in itself, and arranging the working schedule for the twelve months of the year on that basis.

Professor Finch summarizes† the need of instruction in aerial mapping methods in the engineering courses of the universities. This is, of course, to be desired. Research work in surveying, both in aerial methods and ground methods, would seem to be a proper function of technical schools, and here, again, the writer would suggest that improvements in ground surveying methods are equally desirable. The point is that better means of securing proper final results in surveying are to be sought, rather than the perfection of any one particular method to the exclusion of others.

Mr. Wells related‡ an experience in North Adams, Mass., in which the city survey never got beyond the triangulation system. The suggested reason for this is that the public did not understand the use and value of the work up to that point, and, therefore, support for its continuation was not forthcoming. This is an all too common situation. The writer's only thought is that every attempt should be made to focus public attention on the ultimate

* *Proceedings, Am. Soc. C. E.*, January, 1929, Papers and Discussions, p. 240.

† *Loc. cit.*, p. 243.

‡ *Loc. cit.*, p. 242.

use of the survey information—that is, city planning, sewer and water improvements, etc.—rather than on the survey itself. It is practically impossible to secure public understanding of a survey program; but the plans and designs made possible by survey information are easily understood.

In conclusion, the writer would like to reiterate that the great need in American civic affairs is for better planning and better operation of all public improvements and facilities. For this, it is necessary that basic facts relating to the land and its occupation be matters of record and readily available to both public officials and citizens. Experience demonstrates that this basic information is nearly enough alike in all cities to justify the acceptance of a uniform standard of required survey information. An account of this survey schedule, and the methods used in executing it, is the purpose of the paper and the subsequent discussion.

At present, he is inclined to agree with Professor Finch that the showing of the survey information in the form of a map is the most effective means of presenting it to the public. Mr. Vedder suggests a very practical means of obtaining the information and transmitting it to the city survey through funds raised for assessment maps. The cost of such a map is a logical charge against assessment maps, and it may be pertinent to state also, that the entire city survey is a logical and logical charge against sewer, highway, and water improvement projects. As to the procedure of being topographic mapping during winter in the northern cities, this is a question of policy which best may be settled by the particular conditions applying in any particular city. As a general thing, winter does not interfere with the economical procedure. It seems probable that more mapping may be done for a given expenditure by treating the mapping program as a project in itself, and arranging the working schedule for the twelve months of the year on that basis.

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CONCRETE PAVEMENT OVER POOR SUB-GRADE AT PORT NEWARK, NEW JERSEY

Discussion*

BY CHESTER MUELLER, JUN. AM. SOC. C. E.†

CHESTER MUELLER,‡ JUN. AM. SOC. C. E. (by letter).§—The discussion by Mr. Hammatt,|| contributing some experiences with the soil around San Francisco Bay, is a welcome addition to the paper. One point discussed by Mr. Hammatt, namely, his reference to tile drains,¶ reveals a possible lack of clarity in the paper which this opportunity may serve to correct.

Tile drains were placed beneath the pavement for a twofold purpose, the first of which was accomplished in obtaining a dry sub-grade upon which to proceed with the construction work. As mentioned in the paper,|| the second purpose, that of aiding under-drainage, is not so evident. It is believed, however, that with the type of soil encountered, and with the layer of cinders interposed between it and the pavement, no harm is done in running the water to center drains (Fig. 5**) instead of intercepting it under the pavement edges. At any rate, in the case of the 100-ft. concrete pavement, drains placed 10 ft. inside the edge of pavements (Fig. 6**) should function as well as if they were placed directly beneath the edges because of the shaped clay sub-grade; that is, water will flow beneath the pavement edges and for a distance of 10 ft. to the drains at a relatively rapid rate, with little percolation through the clay, because of the slope of approximately $\frac{1}{2}$ in. per ft. Aside from the question of their contribution to the under-drainage system, the value of the drains in facilitating construction alone was worth their cost.

* Discussion on the paper by Chester Mueller, Jun. Am. Soc. C. E., continued from January, 1929, *Proceedings*.

† Author's closure.

‡ Prin. Asst. Engr., Dept. of Public Affairs, Newark, N. J.

§ Received by the Secretary, March 26, 1929.

|| *Proceedings*, Am. Soc. C. E., January, 1929, Papers and Discussions, p. 249.

¶ *Loc. cit.*, p. 250.

** *Loc. cit.*, September, 1928, Papers and Discussions, p. 2054.

An examination of the pavement after three years shows no reason at this time for suggesting any radical departure from the design adopted in 1926. Elevations taken in February, 1927 (every ten joints at each slab corner), were checked in August, 1928, and it was found that a general settlement had occurred. The differences in elevations were principally between 0.1 and 0.3 ft. At several slab corners and joints the inequality of the settlement has caused noticeable depressions. Despite the stresses set up as a result of this unequal settlement, the slabs have neither cracked nor failed otherwise. Investigation of the tile-drain outlets shows the drains to be functioning during and after periods of wet weather, even if it is believed that the unequal settlement has re-acted unfavorably on the grades of the drainage system. Since the settlement in the paving area was so general, it can only be ascribed to a major movement of the sub-grade soil layers and not to the failure of the under-drainage provided.

Judging by its riding qualities and by the absence of signs of slab failure, it can be stated, in conclusion, that thus far the pavement has fulfilled all expectations.

BY CHESTER MUELLER, JR., AM. SOC. C. E.

CHESTER MUELLER, JR., AM. SOC. C. E. (by letter)—The discussion by Mr. Thammatt, contributing some experience with the soil around San Francisco Bay, is a welcome addition to the paper. One point discussed by Mr. Thammatt, namely, his reference to the drains, reveals a possible lack of clarity in the paper which this opportunity may serve to correct. The drains were placed beneath the pavement for a twofold purpose, one of which was accomplished in obtaining a dry sub-grade upon which to proceed with the construction work. As mentioned in the paper, the second purpose, that of aiding under-drainage, is not so evident. It is believed, however, that with the type of soil encountered, and with the layer of clay intervening between it and the pavement, no harm is done in running the water to water drains (Fig. 5**) instead of intercepting it under the pavement edges. At any rate, in the case of the 100-ft. concrete pavement drains placed in it, inside the edge of pavements (Fig. 6**) should function as well as it they were placed directly beneath the edges because of the shaped clay sub-grade that is water will flow beneath the pavement edges and for a distance of 10 ft. to the drains at a relatively rapid rate, with little percolation through the clay because of the slope of approximately $\frac{1}{4}$ in. per ft. Aside from the question of their contribution to the under-drainage system, the value of the drains in facilitating construction alone was worth their cost.

* Discussion on the paper by Chester Mueller, Jr., AM. SOC. C. E., continued from January, 1929, Proceedings.

Author's closure.

† Print. Asst. Engr., Dept. of Public Affairs, Newark, N. J.

Received by the Secretary March 25, 1929.

Proceedings, AM. SOC. C. E., January, 1928, Papers and Discussions, p. 253.

Proc. cit., p. 250.

** Proc. cit., September, 1928, Papers and Discussions, p. 2024.

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THE PRACTICAL UTILITY OF HIGHWAY TRANSPORT SURVEYS

Discussion*

By W. W. CROSBY, M. Am. Soc. C. E.

W. W. CROSBY,† M. Am. Soc. C. E. (by letter).‡—Concerted and noteworthy surveys to develop the relations between the use of roads and their design and upkeep were made prior to 1924. The 1911 report of the Special Committee of the Society on Materials for Road Construction§ shows that traffic was then recognized as "one of the most important" factors in the selection of the type of construction "considered from both the standpoints of economy and efficiency." In Section 4 of The American Highway Engineers Handbook, the even wider effects of traffic are pointed out and considerable detail as to traffic censuses is included.

In the discussion|| of a paper by Mr. H. R. A. Malleck, on "Construction and Wear of Roads", Mr. W. C. Copperwaithe has referred at some length to the wear as compared with traffic, citing London statistics of the latter, and many other contributors to the discussion recognized the relation of use to design and costs.

Reference to the *Proceedings* of the Association Internationale Permanente des Congrès de la Route will show that, since 1908 when this Association was organized, the relation of "use" to "costs" and other features of highway design and execution has been prominent in the minds of highway engineers and authorities all over the world.

Even Macadam and Telford were alert to the relation of "use" to "results" and, in his "Remarks on the Present System of Roadmaking" (1820), Macadam gave some suggestions as to the establishment of these relations.

* Discussion of the paper by G. F. Schlesinger, M. Am. Soc. C. E., continued from April, 1929, *Proceedings*.

† Cons. Engr., Coronado, Calif.

‡ Received by the Secretary, January 7, 1929.

§ *Proceedings*, Am. Soc. C. E., February, 1912, Society Affairs, p. 75.

|| *Minutes of Proceedings*, Inst. C. E., Vol. CLXXVIII (1908-09), Pt. IV, p. 134.

Probably these scattered references to the extensive bibliography of traffic statistics will suffice to indicate the age of the consideration given to the subject.

The recent traffic surveys in Ohio are, of course, interesting, even if any novelties revealed by them are local, and confirm certain facts or principles indicated elsewhere. It is through what might be called the "integration" or "summation" of such local facts that a proper perspective for the scientific side can be obtained.

The great difficulty in the United States since the beginning of modern road work about 1895, has been that the use statistics concerning highways have always been behind, a kind of a "tail to the dog" of traffic development. Now, that more funds are available for the capture, examination, dissection, or analysis of this "tail", its importance is attracting attention from even others than highway engineers.

In some quarters there seems to be a tendency to make "the tail wag the dog"; that is, some argue that all features of a highway design should be based solely on actual or estimated traffic figures. The writer still argues that, of course, proper design demands consideration of traffic data, but he contends, "in and out of season", that after such consideration should come thought for perhaps more immaterial things. As the standards of living rise, pleasure and comfort make their demands for consideration, in addition to those of economy.

Worthy surveys to develop the relations between the use of the highway and upkeep were made prior to 1924. The 1911 report of the Special Committee of the Society on Materials for Road Construction shows that traffic was then recognized as "one of the most important" factors in the selection of the type of construction "considered from both the standpoint of economy and efficiency." In Section 4 of The American Highway Engineers Handbook, the even wider effects of traffic are pointed out and considerable detail as to traffic censuses is included.

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Evan Macnaman and Tenford were alert to the relation of "use" to "results" in his "Remarks on the Present System of Roadmaking" (1920). Macnaman gave some suggestions as to the establishment of these relations

* Discussion of the paper by G. F. Schuster, M. Am. Soc. C. E., continued from April, 1920, Proceedings.

1. (Cont. Engrs. Colorado, Calif.)

1 Received by the Secretary, January 7, 1920.

2 Proceedings, Am. Soc. C. E., February, 1912, Society Address, p. 12.

3 Minutes of Proceedings, Inst. C. E., Vol. CLXXVII (1902-03), Pt. IV, p. 124.

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LETTING CONSTRUCTION WORK BY COMPETITIVE BIDDING

Discussion*

By MESSRS. J. A. LENECEK, JACOB S. LANGTHORN, AND EDWARD W. BUSH.†

J. A. LENECEK,‡ M. Am. Soc. C. E. (by letter).§—The author has done a great service by offering a list of suggestions of a very practical value. It is regrettable, however, that this admirable paper suffers somewhat by the author's occasional criticism of the engineer's work through the different phases of competitive bidding. Sometimes these critical observations are self-contradictory.

For instance: The author admits|| that "nobody can see very far under water, or into the earth", but, later, he seems to forget that the word "nobody" should impartially include not only the contractor but the engineer as well. This inconsistency is apparent when the author suggests that the engineer should remove all uncertainties from sub-surface work, in advance of receiving bids. According to current practice the engineer supplies the bidder with all the necessary field facts (borings, soundings, etc.) and from these the contractor draws his own conclusion about the probable risk of the proposed work. No doubt an uncertainty always will exist if such field facts are not correctly interpreted by the bidder; but for such a condition the contractor should blame only himself, because it is primarily his affair to know his business. He should not expect any help from the engineer, who is usually busy in his own field. As a matter of fact the competent contractor knows how to analyze his costs correctly, while the inexperienced bidder, as a rule, estimates the risk too highly and on account of his high bid eliminates himself, which is a wholesome condition for the owner.

The author suggests¶ a very good method of testing the bidder's financial responsibility, but it hardly offers any hint of how to test the contractor's

* Discussion on the paper by Edward W. Bush, M. Am. Soc. C. E., continued from April, 1929, *Proceedings*.

† Author's closure.

‡ Section Engr., Board of Transportation of the City of New York, New York, N. Y.

§ Received by the Secretary, March 1, 1929.

|| *Proceedings*, Am. Soc. C. E., November, 1928, Papers and Discussions, p. 2439.

¶ *Loc. cit.*, p. 2453.

competency. Perhaps the author also subscribes to the prevalent idea that a financially responsible contractor is in a position to acquire satisfactory plant and a good organization; however, money cannot buy successful experience and a good reputation.

In Spanish-American countries as well as in certain countries of Continental Europe, it is quite customary to subject the bidders on public work to a competency test. For this purpose a careful engineer's estimate is secretly prepared. Measures are also taken to prevent any information concerning the work from reaching the prospective bidders. To that end the chief engineer of the main Office for Public Works selects a few of his widely separated district offices and assigns to each office the preparation of a certain part of the engineer's estimate. The selection and assignments are known only to the chief engineer. Each district office presents its completed work in sealed envelopes at the public hearing held for the opening of the submitted bids. At this meeting all bids are tabulated. Later, the engineer's estimate is compiled from the sealed envelopes, and finally the successful bidder is selected. His detailed computations then must be submitted.

The fortunate bidder is picked out in a manner quite different from general practice in the United States. The lowest bidder is not chosen, necessarily, but the one whose bid is numerically closest to the engineer's estimate. It is contended that by this method the contractor demonstrates the reliability of his experience and his organization and that he is qualified therefore to do the work. The bidder is also required to prove his financial responsibility. The writer has had an opportunity to observe that the entire system "works" remarkably well.

JACOB S. LANGTHORN,* M. A. M. Soc. C. E. (by letter).†—The author's long experience in the surety phase of contracting has enabled him to produce a very useful paper on the business side of engineering. A development of similar interest is the formation in 1927 of a Board of Reference in the City of Boston. This body is composed of twelve members, selected five by the Boston Society of Architects, five by the Boston Building Congress, and two by the Boston Society of Civil Engineers. It is aimed specifically to meet some of the injustices mentioned in the paper.

In service, the Board comes into operation in case a contractor finds items in plans or specifications that seem unfair or inadequate. He then refers the question confidentially to the Board before the competitive bids are submitted. The Board on its part will pass on the criticism and, if justified, will refer the question to the author of the plans and specifications. He, in turn, may answer the criticisms satisfactorily; otherwise he will be asked to amend the forms or issue additional explanatory information to cover the omissions or defects. If time does not permit this, he may give assurance of future correction of defects. In case no agreement is reached, the Board will file formal reports with the three bodies from which it is formed. Such cases will constitute the basis for revisions in the Code of Practice which the organiza-

* Cons. Engr., Office of Borough Pres. of Manhattan, New York, N. Y.

† Received by the Secretary, March 30, 1929.

tions will be asked to approve. It is to be noted that there is no intention of acting in any way after contracts are once made.

As to Principle (9),* it is exceedingly useful to have the date that a blue print is made shown on each copy. Especially is this important in building construction where the architect's tracings are made in pencil. Without the date it becomes difficult to establish priority.

In connection with Principle (14),† one surety company to the writer's knowledge sends periodically a blank form following up work for which it has issued bonds. While the form does not require the complete monthly estimate, it does call for essential information as to the progress, percentage completed, and amount paid to contractor.

With Principle (29)‡ there is a discussion of returning all documents with the bids. This practice is followed by many architects in New York City, but is manifestly unfair, as a contractor is entitled to retain the specifications and drawings upon which he has bid until his bid has been rejected.

EDWARD W. BUSH,§ M. Am. Soc. C. E. (by letter).||—When preparing this paper the writer was frequently puzzled to know which points to include and which to omit. The general subject is broad and many important matters would have been included if the paper already had not stretched itself to a considerable length. Most of the topics are those on which the writer's experience indicated that changes could be made in the average current practice with economic advantages to the construction industry. A few subjects were included that, it is thought, would be of interest in the present-day discussion on the pre-qualification of bidders, the determination of the responsibility of a contractor, etc. These topics are closely interwoven in the general subject.

A few of the discussors have stated that the paper represented the viewpoint of the surety, but the writer disavows this and wishes it to be considered as the effort of an engineer who is drawing on many years of professional experience acquired before becoming connected with a surety company, as well as the experience gained after such connection was made. Some of the arguments and principles offered are adverse to what might be termed the best financial interests of the sureties.

The writer greatly appreciates the discussions offered. Many of them add supplemental material of distinct value, and all of them are of interest because they reflect the ideas of different persons variously engaged in the industry. Existing statutes will prevent the adoption at certain places of some of the principles even if they may be thought desirable and there will be many cases in which some of them are either not needed or inapplicable. A few of the discussions explain special conditions under which certain principles do not apply.

Mr. Wason points out¶ that small scale drawings referred to in Principle (11) would be expensive and unsatisfactory, while photostatic reproductions

* *Proceedings*, Am. Soc. C. E., November, 1928, Papers and Discussions, p. 2441.

† *Loc. cit.*, p. 2443.

‡ *Loc. cit.*, p. 2450.

§ Engr., Aetna Casualty & Surety Co., Hartford, Conn.

|| Received by the Secretary, March 9, 1929.

¶ *Proceedings*, Am. Soc. C. E., January, 1929, Papers and Discussions, p. 275.

would be simple and inexpensive. The writer termed* them "reduced scale copies of the drawings" and had in mind the regular drawings reduced by some photographic process as is frequently done when they are bound with the contract.

In regard to Principle (12) Mr. Wason states† that quantity surveys of this kind are sometimes used to adjust extras instead of merely as the basis for progress payments, and such a use may do a rank injustice because the data may have been unbalanced. Whenever a contractor unbalances either a unit price bid or a schedule of the kind here considered, he takes a chance that later may work against him.

If the owner states in the contract (in accordance with the usual practice) that a certain percentage will be withheld from the progress payments, it is important to the owner that he observe this provision strictly and that he does not overpay the contractor at any time. Otherwise, he may be compelled to replenish the fund to the extent of the over-payment, for the benefit of those having lienable claims. Whether the method of paying for floor by floor, etc., suggested‡ by Mr. Wason, is feasible without over-payment depends on the way the amounts are predetermined. Certainly, it is an easy way to handle a problem which, in the past, has been the subject of many acrimonious disputes between contractors and owners.

Principle (26)‡ states that bids should be disposed of within 30 days after they are received. Of course, it would be desirable if this period were reduced to the 10 days offered§ by Mr. Wason, but some public-work bodies meet only once a month; then, also, some time is frequently needed in which to finance a proposed construction. Therefore, 30 days were offered more as an outside figure than as a practice to be followed generally. The main thing is to state a time limit and to make this as short as possible.

Mr. Rudolph mentions|| the quantity-estimating agencies at various locations that now offer estimates to contractors and owners, and states that the engineer or architect who prepares the plans is better qualified to furnish such quantities than outsiders. Principle (28) suggesting quantity surveys applies only to building work and the writer had in mind quantity surveys of the kind offered by the owners in England. For some unknown reason the architects of the United States have never shown any enthusiasm for the English practice although contractors have many times pointed out to the owner the economic advantages if this practice were to become established. Engineers follow the practice every time a unit price contract is offered to bidders with the estimated quantities included. It is believed that the independent estimating agencies now operating in this country are the outgrowth of the efforts of groups of contractors to have joint or co-operative estimates prepared for them. A considerable volume of this quantity survey work is now being done—much more, perhaps, than many realize—and, of

* *Proceedings*, Am. Soc. C. E., November, 1928, Papers and Discussions, p. 2442.

† *Loc. cit.*, January, 1929, Papers and Discussions, p. 275.

‡ *Loc. cit.*, November, 1928, Papers and Discussions, p. 2449.

§ *Loc. cit.*, January, 1929, Papers and Discussions, p. 276.

|| *Loc. cit.*, p. 278.

course, this labor really is being paid for by the owners. The writer agrees with Mr. Rudolph that this estimating can best be done, or at least supervised, by the architects who prepare the plans. If the volume increases and the agencies multiply, the architects, no doubt, finally will consider that quantity surveys are of some economic advantage and that their preparation is a matter to be supervised by them.

Mr. Wait's remarks* are of especial interest as they are from a man who is broadly experienced in unraveling construction tangles. He questions, however, the value of the pre-qualification of bidders as to adequate financial strength and experience when he states that there are others not so fortunate nor successful who can carry out the contract, and that some of the biggest public works built to-day are by contractors who were not known to have financial backing or experience. Now and then one hears it said that "any contractor can get a bond no matter how weak financially, or inexperienced." Those behind the scenes, as it were, in the surety business know that such a statement is not true, and surety bonds on important contracts are not issued on behalf of inexperienced or financially weak contractors. Mr. Wait, perhaps inadvertently, gives the information that will reconcile his statement with the one just made by the writer when he says† that "contractors who were not known to have financial backing" are able to handle large public work contracts successfully. No doubt, these contractors obtained surety bonds as public work was mentioned and if they were financially weak the sureties did not issue the bonds until after additional financial strength was put in the organizations. In most such cases, which are of common occurrence in surety underwriting, the fact that additional strength was put in is known to only the contractors, the sureties, and those supplying it. This is why some apparently weak contractors are able to obtain contract bonds.

Mr. Wait makes a valuable contribution to the paper when commenting‡ on the absurdity of making the contractor assume the risk of the accuracy of the engineer's preliminary investigations of the site—work which may have cost a large sum. His remarks on the manner in which engineers allow lawyers to dictate what shall be put in the contract, are certainly refreshing.

Mr. White,§ in discussing Principle (13), questions the necessity for the owner to withhold a percentage of progress payments made to the contractor when the owner already has the protection of a contract bond guaranteeing the completion of the project. It is true that the bond extends this protection to the owner, but the present bond premium charges are based on the assumption that the customary 10 to 15% will be retained by the owner and if future practice materially reduces the retainage or discontinues it there is every probability that premium charges on contract bonds will be increased. There is a considerable volume of private work on which no bonds are required and the owners in such cases certainly need the protection of retainages.

The retainage often furnishes a good reason why the contractor rushes a job to completion and while it might be advantageous to the owner and

* *Proceedings, Am. Soc. C. E., February, 1929, Papers and Discussions, p. 517.*

† *Loc. cit., p. 518.*

‡ *Loc. cit., p. 519.*

§ *Loc. cit., January, 1929, Papers and Discussions, p. 278.*

the contractor in some cases to make all progress payments in full, it is believed that the time-honored custom of retaining from 10 to 15% is one that is generally desirable. A stipulation of full payments would require the monthly estimates to be made much more carefully than under the present practice of making approximate estimates when a retainage is withheld; and as the retainage is something earned by the contractor he can generally borrow against it if he needs the funds. In certain localities the owners permit the contractors to convert the retainage into high-class interest-bearing securities which are held by the owners, and this privilege has much to commend it.

Colonel Waldron, apparently, takes the position that the more the engineer tells the contractor in advance of bidding the higher the bids will be. He states* that "when the contractor knows all hazards to be encountered, he is apt to bid higher than if he has to take a chance here and there." This assumes that the contractor is a rather stupid fellow who has to be told about a hazard in order to recognize it. It is believed that most present-day contractors are fully competent to recognize quickly the hazards and to bid accordingly. If they are not competent to do this, they do not remain long in the business of contracting. Principles (3),† (6),‡ and (25),§ with the supporting arguments, were offered in the paper with the hope that they would show that it is to the economic advantage of the owner to remove uncertainties if possible before offering work for bidding. As far as any interest of the surety on these points is concerned, it is well to keep in mind that the surety does not begin to pay on a claim until after the contractor has spent or pledged all he possesses. The contractor becomes a poor risk for himself before he is a poor risk for his surety.

Mr. White presents|| the contractor "as a human being entitled to justice" and points out that even after some of the uncertainties are removed the contractor has a strong economic factor to evaluate and a big risk to carry when he is engaged on contract work. In antithesis to this, Colonel Waldron states¶ in the usual procedure "the engineer tries to get as much as he can for his employer; the contractor tries to do as little as possible in order to increase his earnings". The writer believes that most engineers and architects are fair-minded, with no desire to "put over" anything on a contractor; and, also, that most of the present-day contractors desire to perform their contracts fully. According to current business ethics it is poor policy for either to do the contrary. The greater proportion of all the construction work done in this country is in the hands of the members of the Society, which it will not knowingly admit a "crook", be he a contractor or an engineer.

Mr. Bernstein** presents an interesting fact when he states that at times contractors on private work are compelled to accept paper in lieu of cash because the owners have failed to finance the undertakings completely. The

* *Proceedings*, Am. Soc. C. E., January, 1929, Papers and Discussions, p. 280.

† *Loc. cit.*, November, 1928, Papers and Discussions, p. 2439.

‡ *Loc. cit.*, p. 2440.

§ *Loc. cit.*, p. 2448.

|| *Loc. cit.*, January, 1929, Papers and Discussions, p. 278.

¶ *Loc. cit.*, p. 281.

** *Loc. cit.*, p. 282.

writer has seen many financial statements of contractors showing large "accounts receivable" items which were uncollectable because the owners could not pay them. This is more apt to happen to a young contractor than to an experienced one because probably the latter has been caught, or is afraid he might be caught, in this way, and has learned to avoid such pitfalls. A somewhat similar topic was presented in the paper under the caption, "Introductory"* and was mentioned in the paragraph preceding Principle (6).†

In commenting on Principles (17) and (25), Mr. Bernstein points out that frequently the engineer lacks sufficient time to prepare complete drawings, etc., but that when sufficient time is available there is little cause for complaint. The observations of the writer, after being in a position to note this point on hundreds if not thousands of cases, leads him to state that there is room, when full time is available, for considerable improvement in the current practice on these details.

Mr. Stearns§ discusses many parts of the paper in an interesting and constructive way, but some comments on his discussion are needed. He mentions|| the advantage of the repetition in the contract documents of a subject like the provision that the general and all sub-contractors must obtain permits, etc. If Principle (11) is observed, the general conditions of a building contract always will be attached to the specifications of any particular trade and a requirement once expressed will apply to all trades.

In supporting his statement that certain "risks must and should be taken by the contractor", Mr. Stearns mentions|| a case in which longer piles were ordered than could be driven and as the number of linear feet driven was the item paid for, the cost of the extra lengths was borne by the contractor. This example is exactly of the kind the writer had in mind when writing Principles (3) and (6). In line with these principles, the owner in the example just mentioned would obtain more economic bids by dividing the pile work into two items, one covering the lengths delivered, and the other the lengths remaining below cut-off; also, by sub-dividing the first item and asking for bids on different probable lengths that will be required. In applying this scheme the engineer and contractor should confer on the probable lengths needed before the piling is ordered; then the contractor can prepare his bid with a reasonable assurance that he will not run into some unknown bottom condition that will cause him a loss and, therefore, can bid closely without a considerable factor added for the unknown. The writer has used this method of buying pile foundations and knows it to be successful.

Principles (7)¶ and (8)¶ were included by the writer because they are germane to Principles (18)** and (19)**; and also because in recent years the

* *Proceedings*, Am. Soc. C. E., November, 1928, Papers and Discussions, p. 2438.

† *Loc. cit.*, p. 2440.

‡ *Loc. cit.*, January, 1929, Papers and Discussions, p. 282.

§ *Loc. cit.*, February, 1929, Papers and Discussions, p. 519.

|| *Loc. cit.*, p. 521.

¶ *Loc. cit.*, November, 1928, Papers and Discussions, p. 2441.

** *Loc. cit.*, p. 2445.

industry has been offered some discussions on bonds which overlooked many of the points mentioned in the paper. Mr. Stearns is correct in declaring* that Principle (7) is a mere statement of fact—a fact that many States have overlooked when seeking to reduce the cost of contract bonds.

Regarding retainage mentioned* by Mr. Stearns, the writer believes Principle (13)† is true irrespective of whether or not a surety protection is obtained by the owner. Many border-line cases would not have been bonded by sureties if no retainages were held by the owners.

The writer fails to understand why an engineer or architect is frequently so reluctant to give the prospective bidder some idea in the advertisement as to the probable bid price. The engineer would tell the bidder verbally when he is starting his investigation or he would tell newspaper reporters preparing an article on the improvement. The bidder knows the size of job that will fit his organization and nobody is benefited when he bids against his best interests. Why lead him to investigate a \$100 000 job when he is looking for one of \$500 000, or *vice versa*? The sureties are constantly receiving many preliminary guesses on probable bid prices that are from 50 to 100% in error. Such guesses are a disturbing factor in underwriting border-line contract bond cases and have caused much "grief" to both contractors and their sureties. The easy and obvious way is to tell the prospective bidder at the start the approximate price of the contract offered for bids.

Mr. Stearns does not believe the "bidding security"‡ is in fact what the words imply. In nearly every case the contract documents precisely state the conditions under which the security is deposited—that it is a guaranty the contract will be executed, etc., if the bidder is awarded the contract. If the owner feels that he needs a "bidding security" he should get it large enough to cover the probable difference or spread between the bid that is accepted and the next higher even when these two bids are somewhat out of line. Taking Mr. Stearns' viewpoint of the matter, the 3 to 8% mentioned by him is not large enough to show sufficient working capital to finance the job and absorb an ordinary loss, especially as the contractor may have other contracts under way or may take subsequent contracts.

The writer is very much in accord with the closing paragraphs of Mr. Stearns' discussion and believes the paper could have included with propriety another principle on clarity and consistency. Many forms plainly show the confusion brought about from the "scissors and paste-pot" method of preparing them. The writer recalls a contract and specifications wherein the parties of the first and second part mentioned in the contract were transposed in the specifications so that among other things the owner was obligated to pay the contractor \$25 per day for any delay in completing the contract beyond the date set.

Mr. Consoer offers§ many valuable suggestions among which are some on specific data that should be given to prospective bidders, which are in general

* *Proceedings, Am. Soc. C. E., February, 1929, Papers and Discussions, p. 522.*

† *Loc. cit., November, 1928, Papers and Discussions, p. 2443.*

‡ *Loc. cit., February, 1929, Papers and Discussions, p. 524.*

§ *Loc. cit., January, 1929, Papers and Discussions, p. 285.*

agreement with Principles (3), (6), (17),* (21),† and (25). His practice of giving bidders a copy of the "Instructions to Inspectors" no doubt has prevented many disputes during the construction period and is one that could be followed with advantage to all parties.

Mr. Stephenson refers‡ to Principle (21) and discusses whether or not an approximate estimate of the probable bid price, within limits, should be included in the advertisement. In connection with the "spread" between the high and low bids on an ordinary class of work and in cases where the highest bid is perhaps 50% greater than the lowest, the writer has generally considered that the top one-third or one-half of the bidders really did not expect to obtain the job but merely wanted to take it at very attractive prices. The discussion preceding Principle (30)§ considers this class of complimentary bidders and the disturbances they sometimes cause. Of the remaining bidders, a check-up of the volume of other work on hand, the financial condition, the previous experience on similar work, and whether or not new plant must be purchased and additional organization secured by the respective bidders will show which of them really bid closely to obtain the job. In general, it is the bottom one-third of the bidders who, perhaps, are best equipped, who need the work and are seriously competing for it. There is a variation to the above that occurs now and then wherein the talented contractors—those who really know what the job is worth—are found grouped considerably above the lowest bidders and it is evident that the latter are inexperienced and have made a sorry mess in their estimation of the price. Still another variation is that of the very capable contractor who, apparently, bid so far below the others that he faces a loss, but who at completion retains a fine profit after having given the owner a good job.

Professor Kirby offers|| a good summary of the general subject treated by the paper. His comment on Principle (39) questions whether the engineer should go far into the analysis of the contractor's financial condition, stating that this, perhaps, is within the scope of the surety's service. On work which is not bonded the engineer or architect will probably make the analysis if one is made, but on bonded work the surety will make the analysis, not as a service to the owner, but for its own protection. The writer included Principle (39) and its argument because they set forth briefly the methods of the sureties in determining the responsibility of contractors, and it was thought that this information would interest engineers and architects, especially in connection with the awarding of contracts that would not be bonded.

Mr. Root¶ contributes an admirable discussion. The writer is in general accord with him and will only comment on one point. He states that small scale (reduced scale) drawings, bound with the contract, serve no useful purpose. The writer has reviewed a large number of cases in which the contract drawings, evidently reduced by a photographic process, were either

* *Proceedings*, Am. Soc. C. E., November, 1928, Papers and Discussions, p. 2444.

† *Loc. cit.*, p. 2447.

‡ *Loc. cit.*, January, 1929, Papers and Discussions, p. 287.

§ *Loc. cit.*, November, 1928, Papers and Discussions, p. 2450.

|| *Loc. cit.*, February, 1929, Papers and Discussions, p. 527.

¶ *Loc. cit.*, April, 1929, Papers and Discussions, p. 1049.

bound with the contract or placed in an envelope attached to the back cover of the volume, and in no case has there been any uncertainty as to the details shown. Some of these jobs, running into millions, have been of large dams, bridge foundations, conduits, sewers, etc. The method will operate better with engineering than with architectural constructions, and the writer believes that if once tried the engineer will continue to use it on future work, if possible.

In the statement* that "nobody can see very far under water, or into the earth", the writer attempted to epitomize many reasons why changes will be made in contracts after they are signed. The writer fully agrees with Mr. Lenecek† that "nobody" includes the engineer as well as the contractor, but the writer does not recall that the suggestion is made anywhere in the paper that "the engineer should remove all uncertainties from sub-surface work". In Principle (3) and elsewhere the writer offers the thought that the more the uncertainties are removed, the lower will be the bids, which is quite different. Risks are inherent in certain kinds of work, but every time one of them is removed in advance of the bidding the price is lowered.

Mr. Lenecek states that the writer hardly offers any hint as to how to test the contractor's competency although a way to test his financial responsibility is given. In the discussion preceding Principle (39)‡ the writer uses the word, "responsibility", as including, among other qualities, the financial strength and competency. To be responsible the writer states that§ in addition to being honest the contractor "should have ability as evidenced by his past experience and record; an organization suitable for the proposed work; and sufficient working capital or net quick assets to finance the operations and reasonable additional costs". Principle (33)|| refers to questionnaires and the footnote under "Introductory" mentions the "Standard Questionnaires" which were printed in the March, 1926, *Proceedings* of the Society. The writer knows of no working formula worthy of consideration that could be applied to the information secured on a contractor's past experience, etc., that would determine his competency. The matter, except that of financial strength, is one of individual judgment, because so many of the qualities, while capable of being recognized, cannot be evaluated. No two engineers or architects will give the same weight to the information, although it is to be expected that experienced engineers, architects, and surety underwriters will be found in fairly close agreement on an analysis of the kind of data herein considered.

The writer touches on a legal question here and there in the paper and wishes to state that all these points were checked by consultation with eminent legal counsel broadly experienced in the subject matter.

* *Proceedings*, Am. Soc. C. E., November, 1928, Papers and Discussions, p. 2439.

† See p. 1257.

‡ *Proceedings*, Am. Soc. C. E., November, 1928, Papers and Discussions, p. 2454.

§ *Loc. cit.*, p. 2453.

|| *Loc. cit.*, p. 2451.

AMERICAN SOCIETY OF CIVIL ENGINEERS

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PAPERS AND DISCUSSIONS

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PUMPED-STORAGE HYDRO-ELECTRIC PLANTS

Discussion*

By PAUL L. HESLOP, M. Am. Soc. C. E.

PAUL L. HESLOP,† M. Am. Soc. C. E. (by letter).‡—The author is to be complimented on the painstaking thoroughness with which he has searched for, and brought to light the salient facts on the existing and proposed pumped-storage hydro-electric plants. The data gathered here illustrate the wide variety of conditions under which engineers have considered pumped-storage plants to be worthy of construction. They are not limited to high heads. (See Table 1,§ No. 14, Cornabbia, 80 ft.; No. 36, Hemfurth, 130 ft.; and No. 42, Rocky River, 230 ft.) The storage capacity of the reservoir and the size of the installed machinery cover a wide range of values.

Pumped-storage plants will be justified when, and only when, ordinary storage plants are not feasible or economical in the size of installation desired. An ordinary storage plant will perform all the regulating functions that can be obtained from a pumped-storage plant, with but one exception; that exception, however, is quite worthy of note. Assume that the plant discharges into a tail-water pool in which there is a reasonable storage capacity. In event of a breakdown in a steam plant of the system the pumped-storage plant may be called upon to carry part of the system load, although the reservoir may be at a stage at which withdrawal of water is not desirable. Within the limits of the effective storage in the tail-water pool the water may be pumped back into the reservoir when the steam plant is again in service, which is virtually equivalent to delaying the steam plant breakdown to a more favorable time. Similarly, errors in the hydraulic operation of the system may be corrected within reasonable limits by pumping back the released water. With the ordinary storage plant, water once released can never be replaced as potential energy.

* Discussion on the paper by W. W. K. Freeman, Assoc. M. Am. Soc. C. E., continued from March, 1929, *Proceedings*.

† Senior Engr., Hydro-Elec. Div., United Engrs. & Constructors, Inc., Philadelphia, Pa.

‡ Received by the Secretary, February 28, 1929.

§ *Proceedings*, Am. Soc. C. E., November, 1928, Papers and Discussions, pp. 2464-2465.

Pumped-storage plants in the United States will find their greatest field for consideration in the lower reaches of the various rivers, where the industrial activity is greatest, where real estate is costly, and where the valley is likely to be built up with railroads and industrial plants, making the cost of storage on the main river prohibitive. On such a location, near an important load center, a pumped-storage plant may be used to furnish peak and reserve capacity, that is, if a suitable site can be found.

In the United States pumped-storage hydro-electric plants will often have to compete with somewhat antiquated steam plants and furnish power as cheaply as the operating cost (without fixed charges) of such plants.

The author's statement* that the pumping capacity of the Rocky River plant is now double that originally authorized has an interesting explanation. The additional capacity was installed to reduce the time of initial filling of the reservoir and thus shorten the period before the plant would be ready for operation.

The Rocky River pumps are of the vertical-shaft, single-inlet, single-stage, bottom-suction, volute type. Prior to the construction of these pumps, a model was built and altogether twelve different combinations of impellers and suction cones were made. The pumps are by far the highest in power of the single centrifugal units in America and are, it is believed, the largest high-head pumps of any type in the country. One cannot tell from appearances that they are not hydro-electric generating units. Each pump has a suction elbow that resembles the familiar elbow-type turbine draft-tube. Each is capable of delivering 250 cu. ft. per sec., 112 500 gal. per min., or 162 000 000 gal. per day, against an operating head of 240 ft.

* *Proceedings, Am. Soc. C. E., November, 1928, Papers and Discussions, p. 2463.*

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HORIZONTAL CONTROL IN SURVEYING

PRELIMINARY REPORT OF THE COMMITTEE OF THE SURVEYING AND MAPPING DIVISION ON HORIZONTAL CONTROL

Discussion*

BY HORACE ANDREWS, M. AM. SOC. C. E.

HORACE ANDREWS,† M. AM. SOC. C. E. (by letter).‡—Under the sub-heading, "Datum for Surveys of Small Areas",§ it is remarked that the fundamental plane of city surveys is assumed tangent to the Clarke spheroid. In Section (5) of the "Conclusion",|| it is also stated that the survey should be referred to the North American Datum, this datum having been defined previously.

It seems highly probable that surveys of the precision assumed in this report will be almost exclusively those of cities, and the question might be raised as to the North American Datum being the most appropriate one for all the cities in the United States.

On account of general comparability it would be well if all triangulation were reduced to the same horizon which should be that of sea level, but for the ends of local surveys it would be more rational if the mean height of the land were taken as the horizon—or base surface of reference. With places of significant height above sea level the reduction of distances to that level would be so material that even the ordinary surveyor with his customary methods and usual precision of measurement would appreciate them. For an altitude of 1 000 m., a distance measured horizontally of 1 000 m. would need a reduc-

* This discussion (on the report by George L. Hosmer, M. Am. Soc. C. E., for the Committee of the Surveying and Mapping Division on Horizontal Control, presented at the meeting of the Surveying and Mapping Division at Washington, D. C., April 26, 1928, and published in November, 1928, *Proceedings*) is printed in *Proceedings* in order that the views expressed may be brought before all members for further discussion.

† Albany, N. Y.

‡ Received by the Secretary, February 18, 1929.

§ *Proceedings*, Am. Soc. C. E., November, 1928, Papers and Discussions, p. 2498.

|| *Loc. cit.*, p. 2499.

tion of 0.16 m., which would be appreciable even for rough work.* In a city of notable elevation like Denver, Colo., for example, it would be found that distances would be subject to a reduction of about $\frac{1}{1000}$ part of their length. It would seem, therefore, more practical for the plane or datum used for local surveys to be taken as that of the mean elevation of the place or city in question. The choice might be made dependent upon the relation between the amount of reduction needed for distances horizontally measured and the permissible error allowed in making such measurements.

For all practical ends the surface of the earth in any locality may be regarded more simply as that of a sphere osculatory to the spheroid and with its mean radius at the locality of the survey. With this assumption it would seem immaterial, when a system of plane co-ordinates is to be used for a city's survey—and such a system certainly ought to be used—whether or not these co-ordinates were in the directions of the cardinal points. A city with a large number of its streets at right angles to one another might be more practically co-ordinated on axes running in the direction of these main systems of streets. The cardinal points are not essential to the precise mapping of the territory and the layout of maps would be facilitated by having the system of streets parallel with the directions of the co-ordinate axes.

* Free translation from Dr. W. Jordan's *Handbuch der Vermessungskunde* 1878, Vol. II, p. 107.

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ECONOMIC COMPARISONS OF VARIOUS TYPES OF ROAD SURFACING

A SYMPOSIUM

Discussion*

By W. W. CROSBY, M. Am. Soc. C. E.

W. W. CROSBY,† M. Am. Soc. C. E. (by letter).‡—Most highway problems are first presented through consideration of surfacing questions. Analyzing these latter, engineers usually find that they are symptoms of something more fundamental.

In their admirable papers, Mr. Everett§ concludes that "the best type of road surface varies directly with the amount of traffic", while Mr. Kuelling's judgment|| is, after considering two types only, "that the original cost of the surface means very little in comparison with the operating cost" and that the latter is "difficult to compute".

Doubtless, they both will agree that the kind of traffic and its speeds are important factors, as well as such elements as the grades, widths, and location of the highway, in drawing any fair economic comparisons of various types of road surfacing.

Now, if the "road of the future" be considered as one on which the users will have "the right to go farther and faster, carry bigger loads, have luxuries such as lights, be free from stops, and have better policing and greater safety", does this not suggest a super-"major traffic route" and perhaps even a toll road; or is it so far from that actuality as to warrant neglecting consideration of such conditions for surfacings; and on such roads will not the problems of economic comparisons be more simple and certain of solution?

* Discussion of the Symposium on Economic Comparisons of Various Types of Road Surfacing continued from March, 1929, *Proceedings*.

† Cons. Engr., Coronado, Calif.

‡ Received by the Secretary, March 7, 1929.

§ *Proceedings*, Am. Soc. C. E., November, 1928, Papers and Discussions, p. 2431.

|| *Loc. cit.*, p. 2489.

In 1926 the agenda of the Fifth International Road Congress at Milan, Italy, included a resolution concerning "Roads Especially Reserved for Motor Cars", as follows:

"The creation of roads reserved for the use of motor traffic may be considered as justified when the mixed traffic on the ordinary highways * * * leads to a saturation and a deadlock dangerous to the circulation and contrary to transport economy. It may also be justified when the absolute prevalence of motor traffic of every kind * * * renders it necessary to insure obtaining the highest possible return in the form of speed, non-stop running, and safety."

The writer was asked to comment on the propriety of the proposed resolution prior to its consideration and discussion. He saw no serious objection to it from the American point of view, but having in mind some past experiences with "turnpikes" in the United States, he suggested that, in the development of the details under the general resolution, care should be taken to provide for the surrender of the property to the State after a reasonable time allowed for the private capital invested to recoup itself.

However, when the matter came up formally in the Congress, the British and American official delegates refused to agree to the passage of such a resolution, and it was not pressed beyond this point.

The possibility of special motor roads, on which tolls are charged for the privilege of enjoying the advantages, forms a subject that crops up in highway discussions along with the topic of a return to toll bridges, etc.

The writer cannot agree with those who argue that only the public authorities should build roads, bridges, ferries, tunnels, and other traffic facilities for public use and benefit. Many of the reasons against such a position are the same as those advanced effectually against Government operation of railroads, for instance. "Bigger and Better Bureaus" in Government activities is a slogan the use of which may very properly be limited.

He believes that there are many instances in which public interest in transportation facilities will be quickest and best met by private capital and unofficial enterprise and ingenuity. Opportunities for this form of development are increasing on a steeper curve now than for some years past.

On the other hand, the writer has had experiences with turnpike and toll-bridge "rights" which, in their barest essential skeletons, have stood across the path of development of many State highway systems. This convinces him of the need for protection now against any similar situation in the future.

There seems, however, ample opportunity for a safe course between "Scylla and Charybdis", which course, if properly platted and published, may offer an avenue of great public as well as private benefit in the earliest future.

economic comparisons be more simple and certain of solution?

* Discussion of the Symposium on Economic Comparisons of Various Types of Road Surfacing continued from March, 1926, Proceedings.

Trans. Amer. Soc. Civ. Engrs., Vol. 11, 1926.

Received by the Secretary, March 7, 1926.

Proceedings, Am. Soc. C. E., November, 1925, Papers and Discussions, p. 2421.

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AMERICAN SOCIETY OF CIVIL ENGINEERS

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PAPERS AND DISCUSSIONS

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RELATION OF THE LANDSCAPE ARCHITECT TO THE ALLIED PROFESSIONS ENGAGED IN CITY PLANNING

Discussion*

By JOHN NOLEN, M. Am. Soc. C. E.

JOHN NOLEN,† M. Am. Soc. C. E.—The question of the relationship of technical men in getting the world's work done is partly a problem of understanding a little better the different parts that are played by the constituent elements and the possibilities of organization, which are not yet fully comprehended. It would be a mistake to catalog individuals and to think that they can stand only for one part or for one phase of this task.

There is a question of how to organize what might be called efficiency—although that is not exactly a satisfactory word. Two quotations which seem to conflict with each other, might be given with reference to that question because they lead to a determination of the truth.

One is from Herbert Spencer, who says, "Nothing worth while was ever done by a group." It seems to be true that no great reform was ever started, except by an individual. The origin of things comes in single minds and almost at single moments. In an essay entitled "Circles", Emerson says, "When the great man comes, a new order is established." That is certainly true and it is important in plans of organization, especially in connection with city planning, to avoid merely making compromises. There is need to provide opportunity for imagination. English town planners are working hard on that problem—to find ways to set the imagination free and develop in individuals, the capacity to design.

Recently Arthur E. Morgan, M. Am. Soc. C. E., asked a member of the staff of a Research Laboratory what fundamental quality of personality contributed most to accomplishments in that laboratory. The reply was that the controlling limitation is not technical skill or scientific insight, but the capac-

* Discussion of the paper by Arthur A. Shurtleff, Esq., continued from March, 1929, *Proceedings*.

† City Planner, Cambridge, Mass.

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ELASTIC EQUILIBRIUM IN THE THEORY OF STRUCTURES

Discussion*

BY O. G. JULIAN, Esq., AND GEORGE R. RICH, Assoc. M. AM. SOC. C. E.

O. G. JULIAN,† Esq., and GEORGE R. RICH,‡ Assoc. M. AM. SOC. C. E. (by letter).§—The well known and widely used reduction formula,

$$\frac{M}{S} < \frac{f}{1 + \left(\frac{L}{b}\right)^2 \frac{1}{k}} \dots\dots\dots (36)$$

merely considers the unsupported length of the compression flange as a column. The same buckling resistance is assigned to all beams having a given width of flange, irrespective of the "shape factor" (that is, the relation that the stiffness of the member about the axis parallel to bending bears to its stiffness about the axis normal to bending), or the factor of torsional rigidity of the section. The author's rational analysis clearly indicates that the flexural rigidity about both axes and also the torsional rigidity are vital factors in resisting this type of failure.

The critical buckling load for beams of narrow rectangular cross-section and also for I-beams in which the flexural rigidity about the axis normal to the plane of loading is great in comparison with that about the axis parallel to the plane of loading, has been determined analytically for a wide range of loads and terminal conditions by Professor S. Timoshenko|| by considering the changes in potential energy of the elastic system. The writers have attempted to extend Professor Timoshenko's analysis so as to include H-beams, built-up girders, and trusses, loaded either parallel or normal to the

* Discussion of the paper by H. S. Richmond, Esq., continued from April, 1929, *Proceedings*.

† With Jackson & Moreland, D. L. & W. R. R., Hoboken, N. J.

‡ Structural Engr., Jackson & Moreland, Hoboken, N. J.

§ Received by the Secretary, March 9, 1929.

|| "Beams Without Lateral Support," *Transactions, Am. Soc. C. E.*, Vol. LXXXVII (1924), p. 1247.

major axis, and to develop practical working formulas for such members which would not penalize them unduly as appears to be done by the formulas in common use. The values so obtained for critical loads check the author's values fairly closely.

According to Professor Timoshenko's theory* as understood by the writers, it is required to evaluate the following changes in potential energy, or work done, at the instant buckling and twisting occurs:

- V_1 , due to bending about the axis normal to the plane of loading;
- V_2 , due to the bending of each flange or chord about the axis parallel to the plane of loading;
- V_3 , due to twisting of the section; and,
- V_4 , due to the movement of the loads in their plane of application

In so far as buckling and twisting are concerned, as long as $V_1 + V_2 + V_3 > V_4$, the member is in "stable elastic equilibrium" and as load is added it will finally fail when the $\frac{M}{S}$ stress exceeds the yield point of the material.

When $V_1 + V_2 + V_3 = V_4$, "neutral elastic equilibrium" exists, and as further increments of load are added, the potential energy of the elastic system is decreased and the member may fail by buckling or twisting.

In that which follows, a cantilever beam subjected to a concentrated load in the plane of one principal axis has been selected for study. However, the method appears to be general and applicable in the analysis of members subjected to various loads and types of end restraint.

Using the author's nomenclature except as noted; and if W = work done, f = fiber stress in general, and λ = change in length,

$$W = \frac{1}{2} \int f dA \lambda \dots\dots\dots (37)$$

which, provided the elastic limit of the material has not been exceeded, becomes (prior to buckling):

$$W = \frac{1}{2} \int_0^L \frac{M^2 dx}{EI} \dots\dots\dots (38)$$

or, for beams of constant cross-section and homogeneous material,

$$W = \frac{P^2}{2EI} \int_0^L x^2 dx \dots\dots\dots (39)$$

As buckling and twisting occur the load, P , may be resolved into components normal and parallel to the original axis of loading, as indicated in Fig. 13 (c). Then,

$$W = \frac{P^2}{2EI} \int_0^L (1 - \alpha^2) x^2 dx \dots\dots\dots (40)$$

and by subtracting Equation (39) from Equation (40),

$$V_1 = -\frac{P^2}{2EI} \int_0^L x^2 \alpha^2 dx \dots\dots\dots (41)$$

* "Beams Without Lateral Support," *Transactions, Am. Soc. C. E.*, Vol. LXXXVII (1924), p. 1257.

Similarly,

$$V_2 = \frac{P^2}{2 E I} \int_0^L x^2 \alpha^2 dx \dots \dots \dots (42)$$

or,

$$V_2 = \frac{E I}{2} \int_0^L \left(\frac{d^2 y}{d x^2} \right)^2 dx \dots \dots \dots (43)$$

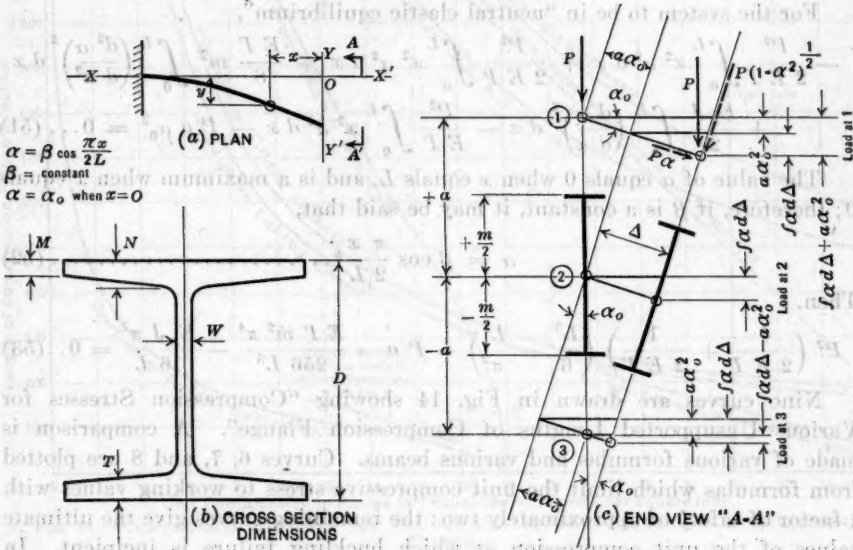


FIG. 13.—THE EFFECT OF BENDING AND TWISTING.

Considering the work done on the flanges separately, it is possible to replace y by $y + \frac{m \alpha}{2}$ for the upper flange and by $y - \frac{m \alpha}{2}$ for the lower flange. Then,

$$V_2 = \frac{P^2}{2 E I} \int_0^L x^2 \alpha^2 dx + \frac{E I}{8 m^2} \int_0^L \left(\frac{d^2 \alpha}{d x^2} \right)^2 dx \dots \dots \dots (44)$$

The work done in twisting a member may be expressed by,

$$V_3 = \frac{1}{2} \int Q d \alpha \dots \dots \dots (45)$$

but, in general, and as given by the author,*

$$\alpha = \frac{Q x}{E_s J} \dots \dots \dots (46)$$

Then,

$$V_3 = \frac{E_s J}{2} \int_0^L \left(\frac{d \alpha}{d x} \right)^2 dx \dots \dots \dots (47)$$

Referring to Fig. 13 (c), it may be seen that,

$$V_4 = P a \alpha_0^2 + P \int \alpha d \Delta \dots \dots \dots (48)$$

* Proceedings, Am. Soc. C. E., January, 1929, Papers and Discussions, p. 7.

but,

$$dA = \frac{P}{EI} \alpha x^2 dx \dots \dots \dots (49)$$

Then,

$$V_4 = Pa\alpha_0^2 + \frac{P^2}{EI} \int_0^L x^2 \alpha^2 dx \dots \dots \dots (50)$$

For the system to be in "neutral elastic equilibrium",

$$-\frac{P^2}{2EI} \int_0^L x^2 \alpha^2 dx + \frac{P^2}{2EI} \int_0^L x^2 \alpha^2 dx + \frac{EI}{8} m^2 \int_0^L \left(\frac{d^2 \alpha}{dx^2}\right)^2 dx \\ + \frac{E_s J}{2} \int_0^L \left(\frac{d \alpha}{dx}\right)^2 dx - \frac{P^2}{EI} \int_0^L x^2 \alpha^2 dx - Pa\alpha_0^2 = 0 \dots (51)$$

The value of α equals 0 when x equals L , and is a maximum when x equals 0; therefore, if β is a constant, it may be said that,

$$\alpha = \beta \cos \frac{\pi x}{2L} \dots \dots \dots (52)$$

Then,

$$P^2 \left(\frac{1}{2EI} + \frac{1}{2EI} \right) \left(\frac{L^3}{6} - \frac{L^3}{\pi^2} \right) + Pa - \frac{EI m^2 \pi^4}{256 L^3} - \frac{E_s J \pi^2}{16 L} = 0 \dots (53)$$

Nine curves are drawn in Fig. 14 showing "Compression Stresses for Various Unsupported Lengths of Compression Flange". A comparison is made of various formulas and various beams. Curves 6, 7, and 8 are plotted from formulas which limit the unit compressive stress to working values with a factor of safety of approximately two; the remaining curves give the ultimate values of the unit compression at which buckling failure is incipient. In obtaining data for Curves 5 and 9, the test load rested directly on the top flange of the beam corresponding to a value of $a = +\frac{m}{2}$. In Curves 1, 2, 3, 4, 6, and 7, the value of a was assumed equal to 0; that is, the load was applied at the center of the web. Equation (53) was used to calculate Curves 2 and 4, the values of J being obtained from the formula proposed by Mr. William B. Campbell:†

$$J = 0.4 DW^3 + 0.1 (N + M)^3 (T - W) \dots \dots \dots (54)$$

in which, D , W , N , M , and T are as shown in Fig. 13 (b). The numerical values of J for the curves in Fig. 14 are as follows:

Curve No.	Value of J .
1	2.31
2	2.53
3	0.295
4	0.322

There is substantial agreement between values of J obtained by Equation (54) and those given in Table 1.† Both the author and Mr. Campbell pre-

* "Beams Without Lateral Support," *Transactions, Am. Soc. C. E.*, Vol. LXXXVII (1924), p 1248.

† *Engineering News-Record*, October 11, 1928, p. 542.

‡ *Proceedings, Am. Soc. C. E.*, January, 1929, Papers and Discussions, p. 18.

dictated their values on tests of **I**-beams. It is doubtful whether the results should be extended to include **H**-beams in which the comparatively heavy flanges carry a greater proportion of the torsional stress. Experimental data on the torsional resistance of **H**-sections seem to be entirely lacking.

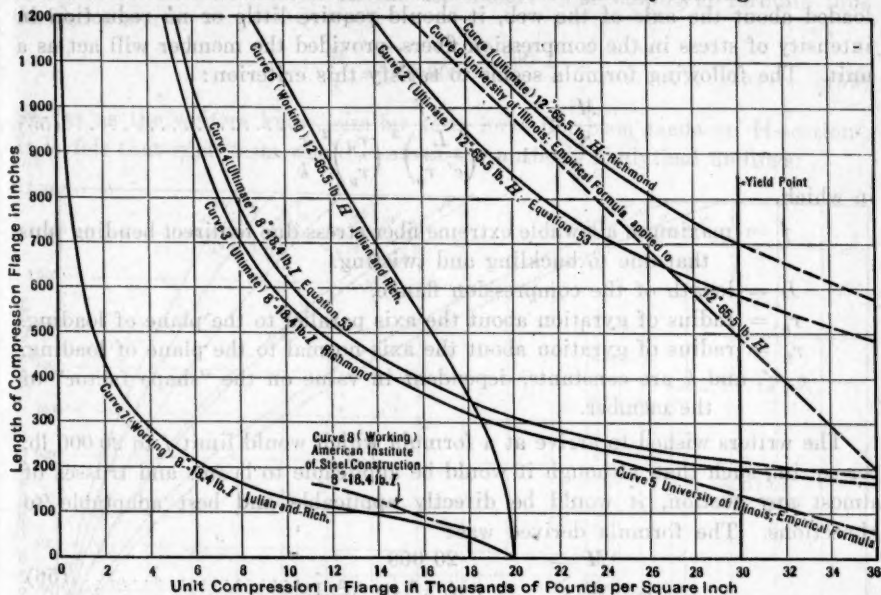


FIG. 14.—COMPRESSIVE STRESSES FOR VARIOUS UNSUPPORTED LENGTHS OF COMPRESSION FLANGE.

As successive increments of load are applied to a beam in bending about the axis normal to the web, the plane form of flexure remains stable up to a certain point; that is, if an accidental lateral force were to disturb the beam, on removal of the force the beam would return to the original plane. As the load is further increased, a sudden lateral deflection occurs, accompanied by torsion. In other words, on application of the last load increment the potential energy of the elastic system decreased, because the last load increment caused a relatively great increase of the work done by the additional dropping of the load as measured by Equation (50). The beam seeks relief by rotation of the section toward a more favorable position as regards lateral components of the loading.

As successive increments of load are applied to the same beam, but in bending about the axis of the web, the plane form of bending remains stable provided the component parts of the beam act as a unit. Failure will finally occur when the yield point of the material is exceeded, because the section is already in the position which affords maximum relief from buckling and twisting and could not possibly rotate toward a more stable position as regards lateral components of the loading.

When tubes and similar sections are subjected to bending no reduction formula for unit compression in the extreme fiber is necessary, because buckling will not occur unless torsional rotation can move the section to a more favorable position.

The author's analysis indicates that an efficient working formula for limiting stress intensity in the compression flange should evaluate the "shape factor" of the section. When members are loaded about the axis normal to the web it should allow higher intensities in **H**-beams than in **I**-beams, and when loaded about the axis of the web, it should require little or no reduction in intensity of stress in the compression fibers, provided the member will act as a unit. The following formula seems to satisfy this criterion:

$$\frac{M}{S} \leq \frac{f}{1 + \left(\frac{L}{c' r_p}\right)^2 \left(\frac{r_n}{r_p}\right)^2 \frac{c}{k}} \quad (55)$$

in which,

f = maximum allowable extreme fiber stress due to direct bending plus that due to buckling and twisting.

L = length of the compression flange.

r_p = radius of gyration about the axis parallel to the plane of loading.

r_n = radius of gyration about the axis normal to the plane of loading.

$c, c',$ and k are constants, dependent in value on the "shape factor" of the member.

The writers wished to arrive at a formula which would limit f to 20 000 lb. per sq. in., such that, although it would be applicable to beams and trusses of almost any section, it would be directly applicable and best adaptable to **H**-sections. The formula derived was:

$$\frac{M}{S} \leq \frac{20\,000}{1 + \left(\frac{L}{4 r_p}\right)^2 \left(\frac{r_n}{r_p}\right)^2 \frac{1}{3}} \quad (56)$$

For **H**-sections having "shape factors" similar to that of a 12-in., 65.5-lb. **H**-beam and loaded parallel to the web, Equation (56) becomes:

$$\frac{M}{S} \leq \frac{20\,000}{1 + \left(\frac{L}{b}\right)^2 \frac{1}{8\,000}} \quad (57)$$

in which, b = breadth of the flange. If the loading is normal to the web, it becomes practically, $\frac{M}{S} \geq 20\,000$.

For **I**-beams having "shape factors" similar to that of an 8-in., 18-lb. **I**-beam and loaded parallel to the web, Equation (56) becomes:

$$\frac{M}{S} \leq \frac{20\,000}{1 + \left(\frac{L}{b}\right)^2 \frac{1}{1\,100}} \quad (58)$$

In Fig. 15, the denominator of Equation (56) has been plotted against L for various sections. The one exception to this is for the curve marked "American Institute of Steel Construction", for which the abscissas are deter-

mined by the quantity, $1 + \left(\frac{L}{b}\right)^2 \frac{1}{2\,000}$

The writers were able to find very little test data on side-buckling of the compression flange and torsional resistance of such sections as **H**-columns. Curve 5 of Fig. 14 has been plotted from tests made at the University of Illinois* of **I**-beams loaded on the top flange. The empirical formula thus derived was:

$$\frac{M}{S} = 40\,000 - \frac{60\,L}{r_p} \dots \dots \dots (59)$$

So far as the writers know, similar tests have not been made on **H**-sections. It is felt that such tests would confirm the author's analytical findings.

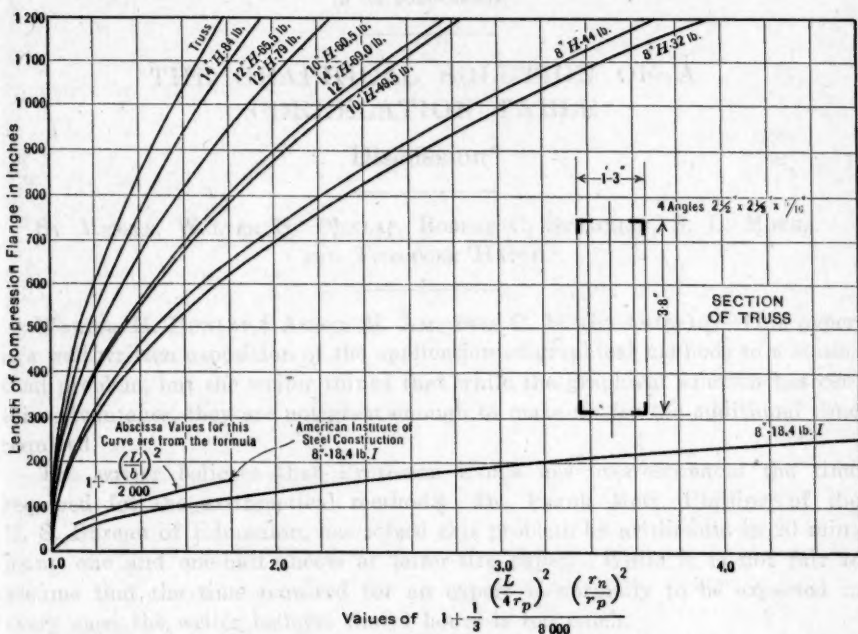


FIG. 15.

The writers wish to express their appreciation of this splendid and timely paper. Structural engineers engaged on railway electrification will owe the author an additional debt of gratitude if he would publish his method of evaluating the resistance of beam sections when subjected to a torsional load, such as that encountered due to "broken wire" loadings.

* "Tests of **I**-Beams in Flexure," by Prof. Herbert F. Moore, *Bulletin No. 68*, Univ. of Illinois Eng. Experiment Station.

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THE GRAPHICAL SOLUTION OF A CORRELATION TABLE

Discussion*

By MESSRS. WALTER H. DUNLAP, ROBERT C. STRACHAN, S. L. MOYER,
AND THEODORE HATCH.

WALTER H. DUNLAP,† ASSOC. M. AM. SOC. C. E. (by letter).‡—This paper is a well written exposition of the application of graphical methods to a statistical problem, but the writer thinks that while the graphical solution has certain advantages, they are not great enough to make up for the additional time required.

The writer believes that Professor Evans has over-estimated the time required for the arithmetical method.§ Dr. Frank McG. Phillips, of the U. S. Bureau of Education, has solved this problem by arithmetic in 20 min., using one and one-half sheets of letter-size paper. While it is not fair to assume that the time required for an expert is normally to be expected in every case, the writer believes that 4 hours is too much.

The advantage of the graphical solution is in the understanding and grasp derived of the problem of correlation while learning about this method of analysis; but after having acquired a fairly good understanding of the method, the regular run of such problems may best be solved by the arithmetical method with the aid of a computing machine. In college the graphical solution is used in the design of a plate girder or a Pratt truss, but the practicing bridge engineer uses the arithmetic method for such a standardized problem. The correlation problem presented by Professor Evans is in much the same category. It is a job to be turned over to an assistant or a computer the same as the computation of the closure and area of a survey of a boundary

* This discussion (of the paper by Weston S. Evans, Esq., published in January, 1929, *Proceedings*, but not presented at any meeting of the Society) is printed in *Proceedings* in order that the views expressed may be brought before all members for further discussion.

† Engr. Cost Analyst, Public Utilities Comm., Washington, D. C.

‡ Received by the Secretary, January 19, 1929.

§ *Proceedings*, Am. Soc. C. E., January, 1929, Papers and Discussions, p. 57.

would be. In fact, some of the computing-machine manufacturers have perfected and are selling a correlation computing machine. However, it is designed for the computation of correlations between time series rather than frequency distributions.

In Professor Evans' problem, the lines of regression are straight and the coefficient of correlation is unusually high. In a problem in which the correlation table clearly indicates a curved line of regression, the writer wonders how the graphical solution would be modified and whether it would show an advantage over the arithmetic solution. For example, suppose the raw regression line (Fig. 12)* in the problem had shown a much more pronounced tendency to curve up at the ends than it does, thus indicating the necessity of fitting a regression line of the form, $y = a + bx + cx^2$, to the data, would the solution of the problem by the graphical method be more advantageous than that by the arithmetic solution?

Three of four places of decimals are quite sufficient for the problem presented, but for partial correlation, additional places are necessary as additional factors are added. For example, if it were desired to analyze the recorded results of many tests of strength of concrete for the purpose of estimating the effect produced by various proportions of cement, sand, crushed stone, water, etc., the methods of partial correlation are indicated as applicable. Partial correlation provides a means of estimating the effect produced by each individual component cause on the assumption that all other causes are held constant.

It is better to take the assumed origin as near as possible to the center of gravity of the table than to place it in one corner for the sake of avoiding negative values. The position of the center of gravity can usually be approximated by inspection; in the author's example it is in the block containing a "frequency" of 24 samples. The advantage is the same as that of assuming the meridian near the middle of a boundary in computing the area; it results in smaller figures with which to work. The correlation table (Fig. 12) in the problem is a comparatively small one. In a table having 18 or 20 or perhaps 25 classes each way, with the total number of cases, N , as much as 1 000, or more, the arithmetic is much simpler if the assumed origin is taken near the center of gravity of the table. One can multiply in his head by 12, but if the multiplier becomes 18 or 23, for example, it is easier to use paper and pencil. By assuming the origin near the center of gravity, virtually all the work down to the computation of the standard deviations can be done in the head, because the multipliers will seldom be greater than 12. For example, in Fig. 12, it will be noted that in the fourth row, $N = 22$, $\Sigma ny = 22 \times 4 = 88$, and $\Sigma ny^2 = \Sigma ny \times y = 88 \times 4 = 352$.

In addition to the text cited by Professor Evans† (which may be regarded as standard), the writer recommends‡ a text by Frederick C. Mills, Associate Professor of Business Statistics at Columbia University.

* *Proceedings, Am. Soc. C. E.*, January, 1929, Papers and Discussions, p. 55.

† *Loc. cit.*, p. 45.

‡ "Statistical Methods Applied to Economics and Business", by Frederick C. Mills, Henry Holt & Co., New York, N. Y.

ROBERT C. STRACHAN,* M. AM. Soc. C. E. (by letter).†—The author lays more stress on methods of obtaining “standard deviation”, “coefficient of correlation”, and “mean regression line”, than on the question, “Why a correlation table?”

As the answer is by no means self-evident, it seems proper to state that—in connection with the theory of probabilities, with its voluminous literature concerning frequency curves, probability curves, frequency surfaces, normal law of error, and kindred subjects—the correlation table or diagram has been developed for the purpose of exhibiting two observed properties or qualities pertaining in different degree to a group of objects, in such a way that the probable influence of either property on the other, may be made apparent.

Thus, the regression lines, Equations (5a)‡ and (7a)‡, or their loci in Fig. 12,§ indicate that if a mortar similar to the tested group has a strength ratio of 80 at 7 days, it should have a ratio of 81.9 at 28 days. On the other hand, for a mortar showing a ratio of 90 at 28 days, the probable ratio is 92.2 at 7 days. These are, of course, expectations founded on the averages represented by the regression lines.

The data chosen by the author to illustrate his method are of a type occurring frequently in engineering work. For example, the effect of temperature on electrical resistance, the influence of percentage of constituent metals on the strength of an alloy, or that of wire-drawing on the elastic modulus of steel, might be studied effectively by similar means.

The author's graphical derivation of the elements essential to the building of the diagram is ingenious, and gives evidence of much study. It also tends to prove that most problems, however complex, may be made to yield to graphics. The desideratum, however, is clearness combined with a simplicity commensurate with the conditions of the problem. Clearness and simplicity appear to the writer to characterize the arithmetical summarization of Fig. 12, rather than Figs. 4 to 10,|| inclusive.

The author estimates¶ that 4 hours should be sufficient for the complete solution of this particular table by either procedure; so that no superiority is claimed for the graphical method on the score of speed. The writer's impression is that if a 4-hour record for this work exists, it must have been made by an exceedingly clever draftsman.

S. L. MOYER,** Esq. (by letter).††—In translating the probable error, r , into a spread between two bounds of a zone of uncertainty, Professor Evans‡‡ attaches a graphic meaning to this elusive index that is most interesting. While the capacity of mathematics to express the degree of credibility may always be subject to question; yet the divergence between the mean regression

* Cons. Engr., New York, N. Y.

† Received by the Secretary, February 4, 1929.

‡ *Proceedings*, Am. Soc. C. E., January, 1929, Papers and Discussions, p. 56.

§ *Loc. cit.*, p. 55.

|| *Loc. cit.*, pp. 50–51.

¶ *Loc. cit.*, p. 57.

** Civ. Engr., Montevideo, Minn.

†† Received by the Secretary, February 14, 1929.

‡‡ *Proceedings*, Am. Soc. C. E., Papers and Discussions, January, 1929, pp. 45 to 57.

lines of X on Y and of Y on X in Fig. 12* does furnish a portrait of the relative accuracy of the different parts of the graph of conclusions, reflecting the wealth or scarcity of the data supporting it.

The procedure involves the method of least squares as applied by Professor Karl Pearson, and his collaborators to the scatter of observations along the two axes of X and Y . As a practical test of the validity of these methods, Professor Pearson's favorite experiment was to examine the scatter of 1 000 shots fired at a line on a target, and these examinations constitute the only positive proof of the departure characteristics presumed to exist. Under this method of interpretation, the trend or true value at which the shots are conceived to be aimed, is missing; and the problem is to discover this line without giving an unwarranted weight to a few scattering values.

The method of least squares was used by Karl Friedrich Gauss as early as 1795, and as later shown by him, it depends for its validity on the Gaussian Law of Error. This law results from a generalization of the binomial theorem known as the multi-nomial theorem. It is, in effect, an indefinite extension of the algebra of a game of dice, in which the numbers or quantities accidentally united, are conceived to add to produce the phenomena or score. This whole mathematical structure, therefore, rests on the premise that the haphazard values of the elemental causes unite by addition to produce the scatter effect.

It seems reasonable to believe that the human, mechanical, and other variations incident to gunfire are most likely to unite, by an addition of the haphazard contributions from these sources, in influencing the scatter of shots on the target. In fact, there seems to be very little question concerning the applicability of the Gaussian law to the reconciliation of the ordinary human and mechanical errors of observation, all of which may be considered as exerting an additive influence on the haphazard combination of causes. In the case of natural phenomena, however, a great number of the known results are multiples of the causes, rather than summations, and it seems worth while to bear this fact in mind as a check on any unwise assumption of accuracy which may be ascribed to a reconciliation of this kind.

The various increments of the chance in the haphazard throw of one die, may be symbolized by the successive terms in the following expression: $x^1 + x^2 + x^3 + x^4 + x^5 + x^6$. This may be called the one-die series, and each term or (x) represents one face of the die or one of the six different, equally likely ways in which one die may fall. The plus signs are merely convenient bounds setting off the divisions between the different kinds of chances or opportunities and have no significance in the chance interpretation, although these signs do serve a mathematical purpose in the determination of the chances for a number of dice in combination. The coefficient of each term of this expression (understood to be unity) indicates the count of ways, while the exponent of each (x) represents the numeral or score on the face of the die. This symbolism means that there is one way to get a score of (1), one way to get a score of (2), one way to get a score of (3), etc., throughout the whole series of numbers carried by the various faces. Assuming that the numerals,

* *Proceedings, Am. Soc. C. E.*, January, 1929, Papers and Discussions, p. 55.

which are accidentally exposed by the throw of two dice, are added to arrive at a score; this sort of coincidence may be symbolized by expanding the one-die series to the second power; that is, by multiplying the same by another such series symbolizing a second die. In this process, the exponent representing the numeral on each face of the first die will be added to the exponent representing the numeral on each face of the second die, and the coefficient of each term of the result will total the count of ways in which each new exponent or additive score may be obtained. This result may be called the two dice series and it reads as follows:

$$x^2 + 2x^3 + 3x^4 + 4x^5 + 5x^6 + 6x^7 + 5x^8 + 4x^9 + 3x^{10} + 2x^{11} + x^{12}$$

This symbolism means that there is but one way to get a score of (2), two ways to get a score of (3), three ways to get (4), etc., throughout the whole series, ending in two ways to get a score of (11) and but one way to get a score of (12). The actual process of carrying out the multiplication of two single-die series to arrive at this result, seems to afford the most illuminating approach to an adequate conception of chance that has ever been devised.

The power to which the single-die series is raised, in this process, symbolizes the number of dice in combination, and this holds true for any number of dice falling in fortuitous coincidence. For four dice in haphazard addition, the chance is symbolized by expanding the single-die series to the fourth power, or the two dice series to the second power. In this expansion, the total of the ways to fall in combination, must always equal the number of faces on one die raised to the power of the number of dice in combination, or in the case of four dice, six to the fourth power of 1296 ways. The resulting scores and counts of ways for four dice scored by addition are shown in Table 2.

TABLE 2.—FOUR DICE IN HAPHAZARD COMBINATION.

By ADDITION:		By MULTIPLICATION:	
Score.	Ways.	Ways.	Score.
4	1	530	1 to 62
5	4	285*	63 to 124
6	10	148	125 to 186
7	20	98	187 to 248
8	35	65	249 to 310
9	56	46	311 to 372
10	80	30	373 to 434
11	104	24	435 to 496
12	125	16	497 to 558
13	140	18	559 to 620
14	146*	5	621 to 682
15	140	12	683 to 744
16	125	4	745 to 806
17	104	4	807 to 868
18	80	6	869 to 930
19	56	931 to 991
20	35	992 to 1 052
21	20	4	1 053 to 1 113
22	10	1 114 to 1 174
23	4	1 175 to 1 235
24	1	1	1 236 to 1 296

* Median or middle of count of ways.

It must be obvious that the dice may be scored by multiplication of the numerals accidentally exposed, without violating any of the dictates of logic or

reason. In order to typify multiple scoring of the dice, it is necessary to violate the strict principles of algebra in the process of expanding to the power of the number of dice. In strictly proper expansion, whenever any two terms are multiplied together, the exponents are added, and this addition symbolizes the addition of these two dice numerals to arrive at a score. It must be apparent then, that the purpose of scores by multiplication may be attained by simply multiplying the exponents in each of these operations, instead of adding. The number of products or ways of combination, in this process, will be exactly the same as the number of sums or ways of combination with exponents added, but the multiplication of exponents or dice numerals gives rise to entirely different scores and radically different coefficients or counts of ways for each sort of exponent or dice score.

In this multiplication of exponents or dice numerals, the least score for four dice is the product of four ones or unity while the maximum score is the product of four sixes or 1296. This range, from 1 to 1296 for four dice in multiplication, is comparable to the range from 4 to 24 for four dice in addition, and in order to show a fair comparison of these two styles of count distribution, the multiple dice scores have been divided into twenty-one classes of magnitudes corresponding to the twenty-one additive dice scores. The count of ways for each of these classes or arrays of multiple scores for four dice have been totaled and given in Table 2 so as to show the relative scatter of these two different styles of haphazard combination. The total of all the different arrays of count of ways will be seen to be exactly the same for either additive or multiple combination (or 1296), while the count distribution has very different characteristics for these two styles of combination.

A comparison of the distribution of ways by haphazard addition, as shown in Table 2, with the count distribution, N' , of observations in Fig. 12 against the two axes of X and Y , suggests that the principles of the Gaussian law may be said to be fairly applicable to the problem illustrated. There are, however, possible limitations to the capacity of the Pearsonian system of correlation that it seems well to weigh. Considering the distribution of the count of ways resulting from the haphazard multiplication of four dice, as shown in Table 2, it becomes apparent that it would be futile to attempt to reconcile this style of scatter by any such methods.

TABLE 3.—FOUR DICE IN A COMBINATION OF QUANTITIES OF COMMON PROBABILITY KIND, OR ALWAYS INVOLVING A COINCIDENCE OF FOUR OF A KIND.

	BY ADDITION (REGENERATIVE).		BY MULTIPLICATION (GEOMETRIC)	
	Score.	Ways.	Ways.	Score.
4 ones.....	4	1	1	1
4 twos.....	8	1	1	16
4 threes.....	12	1	1	81
4 fours.....	16	1	1	256
4 fives.....	20	1	1	625
4 sixes.....	24	1	1	1296

The possibility of causes in multiplication is not the only prospective difficulty which may be encountered. A great number of the contributions to a given result in Nature, in turn, may originate in some common cause or circumstance, and the values combined are thus of the common probability kind instead of the haphazard probability kind; that is, they display a style of combination analogous to a dice score of 4 ones, 4 twos, 4 threes, etc. In the case of elements combining by addition, this results in a magnification of the trend of magnitudes for the original elements, whatever that may be; and in the case of elements combining by multiplication, the results must bear a geometric relation to the sources. These two phases of the possible ways of combination are illustrated in Table 3.

The importance of the distinctions drawn between these different styles of combination, has been previously suggested by the writer* in studies of rainfall and run-off expectancy. These studies are still too fragmentary but, inasmuch as the literature of the subject extending back over a period of more than a century, seems to be unaccountably silent concerning these aspects, it seems wise to call attention at this time to the possible revolutionary effect on correlation conceptions that these distinctions may involve.

THEODORE HATCH,† Esq. (by letter).‡—The engineer is frequently confronted with the problem of determining the relation between two variable measures. There are several methods of doing this, varying in their accuracy from the simple, direct method of fitting the curve by eye, to the more elaborate method of Least Squares. To obtain the equation of the curve, a straight-line relationship between the variables or functions of the variables is usually sought. Although a visual examination of the extent of scatter or deviation of the points from the straight line of best fit gives some idea of the closeness of fit, the degree of relationship is not fixed. This disadvantage is overcome when correlation is made the basis of analysis. This approach possesses several advantages not generally appreciated by engineers.

Two measures are said to be correlated§ when, a series of the first measure of definite sizes being selected, the means of the corresponding sizes of the second measure are found to be a function of the first. If they are independent of the first measure, the measures are said to be non-correlated. A fundamental requirement of this method of analysis is that the relationship be linear, or capable of being rectified to a linear function. Low correlation, therefore, does not necessarily mean absence of relationship; it may imply, simply, the absence of linear correlation. Thus, when $y = kx^2$, there may be low correlation between y and x , but high correlation between $\log y$ and $\log x$, since in the latter form the function is linear. In order to meet this difficulty, a test for linearity has been developed.

By plotting the means of the second measure against the selected sizes of the first, and *vice versa*, two lines known as regression lines are obtained. When

* *Proceedings, Am. Soc. C. E.*, August, 1928, Papers and Discussions, pp. 1845 to 1855.

† Instructor in San. Eng., Harvard Univ., Cambridge, Mass.

‡ Received by the Secretary, March 8, 1929.

§ See "Medical Biometry and Statistics," by Raymond Pearl, W. B. Saunders Co., Philadelphia, 1923.

perfect correlation exists, these two lines are straight and they coincide. When the two measures are mathematically independent, the regression lines assume positions perpendicular to each other. As the degree of mathematical relationship varies from complete independence to perfect correlation, the angle between the regression lines decreases from 90 to 0 degrees. The coefficient of correlation, defined by the author,* finds its basis in this fact. It can be shown that the slope of the regression line of Y on X has the value:

$$b_y = r \frac{\sigma_y}{\sigma_x} \dots \dots \dots (9)$$

and of X on Y ,

$$b_x = r \frac{\sigma_x}{\sigma_y} \dots \dots \dots (10)$$

when $r = 1.0$, b_y becomes the reciprocal of b_x , and the two lines fall together. When $r = 0$, $b_y = 0$ and $b_x = 0$, and the regression lines are perpendicular to each other.

The coefficient of correlation may have any value between $+1.0$ and -1.0 , a positive value indicating a direct relation between the two measures and a negative value an inverse relation. It is easily shown that r can never have a value greater than $+1.0$ nor less than -1.0 . The following arbitrary scale of relationship in terms of the coefficient has been suggested:

Perfect correlation.....	$r = \pm 1.0$
High correlation.....	$r = 0.75$ to 1.0
Considerable correlation.....	$r = 0.50$ to 0.75
Moderate correlation.....	$r = 0.25$ to 0.50
Low correlation.....	$r = 0.25$

An additional measure of the degree of relationship is given by the probable error of the coefficient of correlation. By definition, it is a measure of the reliability of the coefficient itself, this reliability increasing as the ratio of the coefficient to its probable error increases. In practice, the coefficient is not considered to be statistically significant unless it has a value greater than six times its probable error. Thus, a value of $r = 0.3976 \pm 0.0103$ indicates moderate, but nevertheless significant, correlation; whereas a value of $r = 0.7693 \pm 0.2610$ indicates high correlation, but has little statistical reliability, since the coefficient is only three times its probable error.

It should be understood that correlation does not necessarily imply causal relationship. The coefficient of correlation is simply a measure of the mathematical relation existing between two variables. If this fact is kept in mind, many erroneous conclusions may be avoided.

From a knowledge of the coefficient of correlation, it is possible to write the equations of the mean regression lines of Y on X and X on Y , and it can be shown that the curves and equations thus obtained are identical with those given by the Method of Least Squares.

The coefficient of correlation may also be used as a measure of the relative value of various "yardsticks" proposed as indices of a certain condition. An

* *Proceedings, Am. Soc. C. E., January, 1929, Papers and Discussors, p. 52.*

index that does not take into account all factors involved will show low correlation with the condition to be measured; conversely, an index made up of all factors will be highly correlated with that condition.

This point is well illustrated in a recent paper dealing with the comparative values of "cooling power" and "effective temperature" as measures of conditions of comfort in ventilation.* Taking pulse rate as a measure of body comfort, the following coefficients were obtained for the two methods:

Pulse rate and effective temperature.....	$r = 0.7845 \pm 0.029$
Pulse rate and cooling power.....	$r = 0.5632 \pm 0.052$
Difference	$= 0.2213 \pm 0.0595$

These results indicate high correlation between pulse rate and effective temperature and lower correlation between cooling power and pulse rate. Moreover, the coefficient of correlation between effective temperature and pulse rate is twenty-seven times its probable error; whereas in the case of cooling power and pulse rate, the coefficient is only ten times its probable error. Hence, it may be concluded that effective temperature constitutes a better index of body comfort (as measured by pulse rate) than cooling power.

The preceding statements have to do with simple correlation between two related variables. Not infrequently, however, it is necessary to determine the laws of relationship between three or more variables. In laboratory investigations, this is done by controlling certain of the variable factors in each experiment. In dealing with natural phenomena, this procedure is not possible, but it can be done mathematically by methods of partial correlation. The coefficient of partial correlation gives the degree of relationship between two variables, when all others are held constant by the mathematical process, and can be expressed, as follows:

$$r_{12.34 \dots n} = \frac{r_{12.34 \dots (n-1)} - [r_{13.34 \dots (n-1)}] [r_{23.34 \dots (n-1)}]}{[1 - r_{13.34 \dots (n-1)}^2]^{\frac{1}{2}} [1 - r_{23.34 \dots (n-1)}^2]^{\frac{1}{2}}} \dots (11)$$

The subscripts indicate that the correlation is between Measures 1 and 2, with Measures, 3, 4...n held constant. For three variables, the expression becomes:

$$r_{12.3} = \frac{r_{12} - r_{13} \cdot r_{23}}{(1 - r_{13}^2)^{\frac{1}{2}} (1 - r_{23}^2)^{\frac{1}{2}}} \dots (12)$$

The following example will illustrate the application of Equation (12). The effect of altitude and distance from the ocean upon the annual rainfall in San Diego County, California, are to be determined from the data given in the records of the various rainfall stations.† By simple correlation, the following coefficients are obtained:

Rainfall and altitude, $r = +0.8779 \pm 0.2346$
Altitude and distance, $r = +0.8747 \pm 0.354$
Rainfall and distance, $r = +0.6172 \pm 0.0934$

* "Physiological Reactions of Resting Subjects to Cooling Power and Effective Temperature," by J. Argyll Campbell and T. C. Angus, *Journal of Industrial Hygiene*, December, 1928, 10, 331

† Data taken from Mead's "Hydrology," p. 294, McGraw-Hill Book Co., N. Y., 1919.

From these coefficients of simple correlation, it seems that (a) rainfall increases with altitude; (b) altitude increases with distance; and (c) rainfall increases with distance. The increase of rainfall with distance, however, is actually due, not to the effect of distance from the ocean, but to the fact that, with increasing distance, the altitude is greater. To take this into account, the coefficients of partial correlation may be used. These are found to be as follows:

Partial coefficient between rainfall and altitude, $r = +0.887$

Partial coefficient between rainfall and distance, $r = -0.206$

These coefficients show high positive correlation between rainfall and altitude when distance is held constant and rather low negative correlation between rainfall and distance when altitude is held constant. This inverse relationship between rainfall and distance which is obtained by partial correlation is in accordance with the laws of rainfall.

From a knowledge of the coefficients of partial correlation, it is possible to write the equation of the relationship between the variables. For three measures, the equation has the form:

$$X_1 = \left[\beta_{12.3} \frac{\sigma_1}{\sigma_2} \right] X_2 + \left[\beta_{13.2} \frac{\sigma_1}{\sigma_3} \right] X_3 + M_1 - a M_2 - b M_3 \dots (13)$$

in which, X_1 , X_2 , and X_3 are the measures; M_1 , M_2 , and M_3 are their means; σ_1 , σ_2 , and σ_3 are their standard deviations; and

$$\beta_{12.3} = \frac{r_{12} - r_{13} \cdot r_{23}}{(1 - r_{23}^2)} \dots (14)$$

$$\beta_{13.2} = \frac{r_{13} - r_{12} \cdot r_{23}}{(1 - r_{23}^2)} \dots (15)$$

$$a = \beta_{12.3} \frac{\sigma_1}{\sigma_2} \dots (16)$$

$$b = \beta_{13.2} \frac{\sigma_1}{\sigma_3} \dots (17)$$

For this example, when X_1 is the annual rainfall, in inches, X_2 is the altitude, in feet, and X_3 is the distance from the ocean, in miles:

$$X_1 = 0.00796 X_2 - 0.628 X_3 + 15.1 \dots (18)$$

These single examples are cited in order to illustrate the use of correlation in experimental work and investigation and, in particular, to suggest to engineers the possible advantages of applying this method to the analysis of their problems.

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FINAL REPORT OF THE SPECIAL COMMITTEE ON IMPACT IN HIGHWAY BRIDGES

Discussion*

BY SEARCY B. SLACK, M. AM. SOC. C. E.

SEARCY B. SLACK,† M. AM. SOC. C. E. (by letter).‡—In compiling and considering available information on impact stresses in highway bridges the Committee has rendered a distinct service to all engineers concerned. The allowance made for impact in highway bridge design has varied widely, being different in almost every specification. Some engineers have taken the position that the available data were not conclusive and have made no allowance for impact, while others, through probable misinterpretation of the data, have made allowances which appear unreasonably high. The recommendation of the Committee is a happy medium between these extreme positions.

One of the main criticisms which will undoubtedly be made of the report is that the impact allowance for a given span length is the same for all truss members, whereas maximum stresses in the web members are caused by much shorter loads than the full span length; hence, greater impact allowance should be made. To meet this condition, L of the impact formula has frequently been taken as the loaded length instead of the span length, as suggested by the Committee.§ The result of taking L as the loaded length is that the allowance made for impact is uniform for the chord members, but varies for the web member in each panel. From the meager test data available this seems to be a needless refinement and the Committee is to be commended for simplifying the practice by recommending a formula‡ which will give a uniform allowance throughout a given span. A further justification of this course is that the sheer weight of a modern highway bridge with concrete floor and

* This discussion (of the Final Report of the Special Committee on Impact in Highway Bridges, presented at the Annual Meeting, January 16, 1929, and published in March, 1929, *Proceedings*), is printed in *Proceedings*, in order that the views expressed may be brought before all members for further discussion.

† Bridge Engr., State Highway Board of Georgia, East Point, Ga.

‡ Received by the Secretary, March 8, 1929.

§ *Proceedings*, Am. Soc. C. E., March, 1929, Papers and Discussions, p. 735.

paving has a great absorbing inertia, and a large part of the stresses in the members is due to dead load (except of course counters). Varying the impact allowance has comparatively little effect on the members.

In certain cases such as counters or unusual web designs, the formula proposed by the Committee probably does not provide sufficient allowance for impact. These are such special and limited cases, however, that a general formula could hardly be expected to apply to them.

In the interest of uniformity and simplification of highway bridge specifications it is to be hoped that the recommendations of the Committee will be widely adopted.

Report of the Committee on the Highway Bridges
 The Committee on the Highway Bridges, created by the American Society of Civil Engineers in 1906, has the honor to submit herewith its report on the subject of the impact allowance in the design of highway bridges. The Committee was organized in 1906, and has since that time held several meetings for the purpose of considering the subject. It has received many suggestions from members of the Society and from other engineers, and has endeavored to give careful consideration to all of them. The Committee believes that the following recommendations will be of service to the engineering profession in the design of highway bridges.

The Committee believes that the impact allowance should be based on the weight of the vehicle and on the speed of travel. It recommends that the impact allowance be expressed as a percentage of the dead load, and that it be computed by the following formula:

$$I = \frac{W}{1 + 1.25 \sqrt{L}}$$

where I is the impact allowance in per cent, W is the weight of the vehicle in tons, and L is the length of the bridge in feet. The Committee believes that this formula will give a reasonable allowance for impact in all cases. It also recommends that the impact allowance be applied to the dead load of the bridge, and that it be computed for the maximum load on the bridge. The Committee believes that these recommendations will be of service to the engineering profession in the design of highway bridges.

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PROGRESS REPORT OF THE SPECIAL COMMITTEE ON STEEL COLUMN RESEARCH

Discussion*

By HENRY S. PRICHARD, M. AM. SOC. C. E.

HENRY S. PRICHARD,† M. AM. SOC. C. E. (by letter).—A consideration of the progress reports, made in January, 1926,§ and January, 1929,|| by the Special Committee on Steel Column Research, excites admiration for extensive, painstaking, able analyses, and gives ground for hope that much of practical value will result from the research after the contemplated further investigations have been completed. The final report will be awaited with much interest.

The theory of flexure is based on the proportionality of stress to strain and on the modulus of elasticity being constant for the material to which the theory is applied. When it is applied to a body as a whole, it is assumed that the details are properly proportioned. In general, the theory is applied as if friction between the bodies or parts to which it is applied and other bodies or parts is negligible. It is gratifying to find so much verification of theory in the physical investigations within these qualifications. It is also gratifying that the modulus of elasticity for steel proved to be so nearly uniform for all cases observed, and that the mathematical analyses based on proportionality of stress to strain agree with experimental determinations, as far as the general behavior of columns is concerned, over such a considerable range of stress.

Engineers and architects, for many years, have based their practice with regard to beams on the ordinary theory of flexure, and during the same period have based their practice as regards columns on arbitrary formulas, some being

* This discussion (of the Progress Report of the Special Committee on Steel Column Research, presented at the Annual Meeting January 16, 1929, and published in February, 1929, *Proceedings*), is printed in *Proceedings*, in order that the views expressed may be brought before all members for further discussion.

† Analytical Engr., Am. Bridge Co., Pittsburgh, Pa.

‡ Received by the Secretary, March 18, 1929.

§ *Transactions*, Am. Soc. C. E., Vol. 89 (1926), p. 1485.

|| *Proceedings*, Am. Soc. C. E., February, 1929, *Papers and Discussions*, p. 357.

little more than rule-of-thumb methods modified more or less by experiments and experience; some being the result of mistakes in theory; and some being based on pseudo theories or mere notions. Yet the ordinary theory of flexure applies more closely to columns than to beams and, intelligently applied, finds better confirmation in the case of columns than in that of beams. This is due in part to the facts that the ordinary theory of flexure neglects the influence of shear on deformation and shear is more of a factor in beams than in columns. The permanent sets under transverse loads indicated in Column (6) of Table 5* may have been due to the transverse load at the center being almost concentrated on a line. The ordinary theory of flexure, which neglects the influence of shear on deformation, gives no indication, in the cases tabulated, of stresses intense enough to produce permanent sets. When the influence of shear is considered, however, it indicates that very high stresses, much higher than those indicated by the ordinary theory, may occur if the transverse load is too highly concentrated.†

Under ideal conditions a centrally loaded column with frictionless hinged ends would be subjected to no shear; but in order that the various fibers constituting the column shall act together, it must have potential resistance to shear. This potential resistance has been neglected in Euler's formula, in consequence of which that formula gives a result which is, theoretically, slightly too great, but only very slightly so for a column with a reasonable cross-section.‡

Section 35, § advocating "Factor of Safety Method in Column Design", is highly commendable and should receive special consideration. If rational methods of computing the relation between loads and stresses and shears are used, then, actually to obtain, in a column, the factor of safety desired, it must be applied to the critical load and not to the critical stress or shear, the critical load being the load which would produce the critical stress or shear; that is, the stress or shear at or near failure.

Quoting from Section 35:

"* * * the logical process in column design is to select a formula which will as nearly as practicable give values of ultimate strength, or some point which represents the practical ultimate value of the column; then to divide the resulting value of average stress, P/A , by the factor of safety corresponding to that used in other members * * *".

This leaves an open question as to just what point in loading should be considered the critical point to which the factor of safety should be applied. In its First Progress Report the Committee stated|| that:

"The Society's Special Committee on Steel Columns and Struts made use of what was named by it the 'Useful Limit Point' (U. L. P.)¶ as being of more significance in some respects than the ultimate strength of the column or the

* *Proceedings*, Am. Soc. C. E., February, 1929, Papers and Discussions, p. 372.

† *Transactions*, Am. Soc. C. E., Vol. 89 (1926), p. 1257, Equation (22); see, also, Vol. LXXV (1912), pp. 932-953, for effect of shear on deformation of beams.

‡ *Loc. cit.*, Vol. 89 (1926), p. 1423; see, also, *Engineering News*, February 25, 1909, p. 206.

§ *Proceedings*, Am. Soc. C. E., February, 1929, Papers and Discussions, p. 424.

|| *Transactions*, Am. Soc. C. E., Vol. 89 (1926), p. 1487.

¶ *Loc. cit.*, Vol. LXXXIII (1919-20), p. 1618.

yield point of the material. Although there is good ground for adopting this point in the stress-deformation diagram, as the best measure of column strength, it is not practicable to adhere to this method in a study of all available data. Information does not exist to determine the U. L. P. in a great many of the tests that have been made; but, nevertheless, it is believed that the results of such tests are valuable and cannot be neglected."

The Committee further stated:*

"It is possible that some such point as the U. L. P. used by the Society's Special Committee on Steel Columns and Struts would be a more accurate reference point than the yield point, but as no other tests give this information,† it seemed best to use the yield point instead, and it is believed that the diagrams represent fairly the relations indicated.

"The difficulty of securing correct information regarding the elastic properties of the material actually used is well known. The values of yield point as determined by the usual 'drop of the beam' method depend largely on the details of testing; the mill tests vary largely from laboratory tests; and the material varies greatly in thin and thick sections, and at different points in the same section."

The writer's opinion of the critical value of a structural steel member in direct compression or tension, in the formation of which he gave great weight to the report of the Special Committee on Steel Columns and Struts, has been published.‡

In ordinary cases the only criteria which an engineer or architect has of the strength of the steel in the structure he has designed, are the specimen tests, furnished by the mill, of the tensile yield point, as determined by "drop of of beam", and the ultimate tensile strength. The "yield point" as furnished by the mill is often much higher than that found in laboratories, such as that of the U. S. Bureau of Standards, and is usually much higher for specimens than for full-sized structural members.

The U. L. P., the yield point, and the ultimate strength of different rolled sections, as found in properly conducted tests in reliable laboratories, differ greatly for steel nominally of the same quality, even from the same heat. For instance, in the careful tests by Marshall, the "elastic limit", as he terms it, taken by "drop of beam", of a specimen from a 4-in. by $\frac{1}{2}$ -in. flat, was 50% greater than that of a specimen from a 3 in. by $1\frac{1}{2}$ -in. flat, although they were from the same blow of Bessemer steel.§ The extremes in the characteristic U. L. P.'s of the columns made of 60 000 lb. (desired) steel, tested for the Society's Special Committee on Steel Columns and Struts were 19 000 and 35 000 lb. per sq. in., respectively.||

The great differences in utilizable strength due to the thermal and mechanical conditions encountered in the normal processes of manufacture, as distinguished from differences in chemical composition, overshadow the differences due to length, radius of gyration, and form of cross-section, in cases where the

* *Transactions*, Am. Soc. C. E., Vol. 89 (1926), p. 1522.

† In numerous tests contained in various Watertown Arsenal reports the strains near the U. L. P. were recorded at short intervals of stress. In each of such cases, the U. L. P. can be closely approximated.

‡ *Transactions*, Am. Soc. C. E., Vol. 89 (1926), pp. 1236 and 1239.

§ *Loc. cit.*, Vol. XVII (1887), Table I, p. 68(a).

|| *Loc. cit.*, Vol. 89 (1926), p. 1268.

conditions are such that the capacity of the column can be gauged more by its resistance to stress than by its degree of stiffness. In long columns, the intensity of elastic stress which steel will resist is not so much of a factor.*

The members of framed structures, even pin-connected ones, have secondary stresses, in addition to direct stresses. The direct stresses are generally computed as if the structure were perfectly articulated and the differences between the combined stresses and the direct stresses are considered secondary stresses. The determination of the secondary stresses is a complicated matter. It requires for each member a consideration of all the members, including their section and make-up, and, from a practical point of view, is only approximate, on account of the complication due to connections at the ends of the members. Apart from these considerations, as the determination of secondary stresses is based on the proportionality of stress to strain, it can only be correct within the limits to proportionality. If carried beyond these limits it is liable to be very faulty.

In view of the uncertainties as to the intensity of stress to which steel structures are liable in many of their details and of uncertainties as to the elastic resistance to stress possessed by the numerous pieces of steel used, it is fortunate that steel possesses capacity to recover elasticity—albeit with permanent set—after it has been subjected to plastic deformation in service. This phase of the subject should and probably will form part of the research of the Committee before it has concluded its labors.

The many phases involved in column research should be kept in mind in judging the reports of the Committee; otherwise, too much will be expected. Many uncertainties are inevitable under the conditions of manufacture, testing, and use which now exist.

* *Transactions, Am. Soc. C. E., Vol. 89 (1926), p. 1292, Fig. 6.*

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CONTINUOUS BEAMS OVER THREE SPANS

Discussion*

By I. OESTERBLOM, M. Am. Soc. C. E.†

I. OESTERBLOM,‡ M. Am. Soc. C. E. (by letter).§—The response to this paper has been very gratifying; that numerous discussions have been submitted for an extended period would possibly indicate that the question raised has real and timely interest. Generally this has been admitted by the discussors. The purpose and also the contents of the paper have been commented on with approval, and whatever criticism has been submitted has been searching and constructive. Some of it has gone beyond the scope of the paper, but it has been valuable, nevertheless. Thus, from two different impulses, there have been secured supplementary, useful ideas and methods as well as definite information in regard to simplified designs of continuous beams.

Some of the critics have definitely expressed the opinion that a great deal was omitted from the original paper. The question might be raised, however, if a very complete paper on the subject would not have been an unfortunate contribution. It would have reduced the opportunity of supplementary contributions from interested engineers, but even worse than that, it would have entered into the more general field of stresses and strains in elastic frames, already so thoroughly and competently treated by a legion of highly qualified authorities.

It was the purpose of the paper—unfortunately it seems to have been insufficiently emphasized—to reduce to easily workable form some of the very complicated combinations of formulas for continuous beams, so that reasonably correct moment factors might be quickly ascertained for the most common cases of simple continuity.

Perhaps the examples given were presented in a manner which seemed to contradict the purpose of speed in execution. By neglecting variability of

* Discussion on the paper by I. Oesterblom, M. Am. Soc. C. E., continued from February, 1929, *Proceedings*. By an error a part of this discussion was omitted when it was published in April, 1929, *Proceedings*. It is therefore published herein in full for the benefit of the membership.

† Author's closure.

‡ Structural Engr., St. Louis, Mo.

§ Received by the Secretary, February 15, 1929.

load the elements of each moment factor may be summed up into one. This, in most cases, would be warranted either by fact or a very close approximation. The examples should be left as shown, however, because they give very completely and accurately all that is essential for simple cases of continuity within the limits outlined. They are typical of cases with most extensive variations.

For the routine work in a designing office, the computation form shown in Table 13 would possibly serve best. (See Example 1.*)

TABLE 13.—FORM FOR COMPUTATION OF MOMENTS.

	Bending moments.			
Dead load.....	$67 \phi \times 22.5^2 = 34\ 000 \phi$			
Live load.....	$60 \phi \times 22.5^2 = 30\ 500 \phi$			
MOMENT FACTORS (ϕ).				
	Outer span.	Middle span.	Supports.	
Dead load.....	$\begin{matrix} + 0.084 \\ - 0.008 \\ + 0.007 \end{matrix}$	$\begin{matrix} - 0.089 \\ + 0.015 \\ - 0.089 \end{matrix}$	$\begin{matrix} - 0.091 \\ - 0.005 \\ + 0.013 \end{matrix}$	
Total.....	$+ 0.088$	$- 0.063$	$- 0.063$	
Live load.....	$\begin{matrix} + 0.084 \\ + 0.007 \end{matrix}$	$\begin{matrix} - 0.089 \\ - 0.089 \end{matrix}$	$\begin{matrix} - 0.091 \\ - 0.005 \end{matrix}$	
Maximum.....	$+ 0.091$	$- 0.078$	$- 0.096$	
TOTAL MOMENTS.				
Dead load.....	$+ 2\ 980$	$- 2\ 130$	$- 2\ 800$	
Live load.....	$+ 2\ 760$	$- 2\ 360$	$- 2\ 930$	
Total.....	$+ 5\ 740$	$- 4\ 490$	$- 5\ 730$	

A small error occurs, for the support moment only, and this is due to taking the live load at 60 lb. instead of 80 lb. for the center span. The error is less than 1 per cent.

Rapid pencil notations obviously are sufficient, with the multiplications done by slide-rule. The simplest problems thus can be solved in less than 5 min. and the more complicated in less than 10 min.

It has become necessary to emphasize the simplicity and the time element, because so many "simplified methods" have been suggested in the discussion. These are all excellent, and will be referred to in the proper place, but they generally fail to take the final step of producing directly applicable moment factors. While there must be problems which require a great deal of care and

* *Proceedings, Am. Soc. C. E., March, 1928, Papers and Discussions, p. 719.*

a corresponding amount of time, because they are complicated by supplementary restraints and variabilities, the majority of cases should be, and are, elementary enough to be solved by exceedingly rapid methods. Anything less than this would merely induce, as it has in the past, a great amount of rough approximations with corresponding inaccuracies. This is an economic problem; if the cost of designing should run too high, it will be turned into a game of chance.

This is not discounting the many excellent contributions which are of a supplementary nature. They are truly supplementary, because they show how the problems should be solved when they fall beyond the scope of the paper.

Indeed it is difficult to imagine a more graceful, ingenious, and efficient method than that of Hardy Cross,* M. Am. Soc. C. E., or the equally ingenious one by George E. Beggs, M. Am. Soc. C. E., both of which have a general application and each its special, although somewhat overlapping, field of usefulness. Professor Cross' method contains a very slight approximation, but it is entirely too small to consider in view of the exceedingly valuable time-saving procedure.

Professor Cross' method is especially recommended for all cases of continuity, that are not soluble, or should not be solved by means of ready-made moment factors. Professor Beggs' method requires a set of instruments and the manipulation of celluloid models. With the complete equipment at hand its usefulness would be unlimited.

It is particularly gratifying, therefore, that these two methods were introduced by Professors Mylrea† and Parcel,‡ but while this acknowledgment is made with gratitude for the reference and admiration for the methods, it is only fair to state that they offer neither negation nor obsolescence for the writer's contribution.

The paper is elementary in its nature; but none the less it was not intended as a text for junior students. It assumed that the reader would know that there were additional restraints; it also assumed that he would know when the use of the simplified formulas and tables would not be permissible. If the writer took this knowledge for granted erroneously, it has been made very clear by the discussion that there are limits beyond which the formulas and factors should not be used.

As soon as one has to deal with more than three spans, or additional restraints from supporting or supported elements, all possibilities for considerable simplification vanish.

Fortunately, however, the vast majority of continuous beams are of three spans and are freely or almost freely supported. The tables and formulas in the paper are, therefore, an offering to those engineers—and their assistants—who thoroughly know their elasticity problems, but who are none the less looking for a rapid and accurate solution of those that confront them most frequently.

* Available in mimeographed copies for the students of the Univ. of Illinois.

† *Proceedings*, Am. Soc. C. E., February, 1929, Papers and Discussions, p. 484.

‡ *Loc. cit.*, January, 1929, Papers and Discussions, p. 162.

In a sense it may be considered an accident that, from the many methods possible, the three-moment theorem was selected as a basis for the paper. It is more than that, however. It is a tribute to a most graceful and accurate solution of an old problem, which has lost nothing by having attained a classical distinction.

From another point of view it stands out with compelling clarity that the one thousand and one (more or less) mathematical masterpieces analyzing stress and strain in elastic frames have had the practical effect of beclouding the elementary issues and setting up—instead of desired simplicity and accuracy—a chimera of faith, hope, and fantasy, which, unfortunately, has been set in a golden frame by committees of authorities and experts. This evil is international, but, curiously enough, it is subject to worship, nationally.

From this viewpoint the method introduced by Professor Cross is particularly significant. From the mass of formulas that are always necessary he has extracted all the essential factors and eliminated others which have insignificant effects on the result and merely tend to confuse the designer in his mathematical operations.

The question has been raised as to what extent brick walls would serve as a restraint for the floor construction. Seemingly, there can be no mathematical answer. To one who has spent a great deal of time in the field, the restraint appears to be generally small. Sometimes, the walls may be heavy, the brick especially good, and the mortar excellent—any one or all of these—and restraint then may be considered. It certainly should be considered whenever it may introduce an element of weakness, but this is seldom the case with the supported three-span elements. More often the restraint is very small, the reduction of moments would likewise be small, and the proposed formulas and factors would be sufficient. In general, the smaller and simpler structures analyzed in this paper are by far in the majority.

The temperature stresses in three-span construction are largely insignificant, but allowance for them can be made by the usual method, as soon as fundamental proportions have been once established. Mr. Way* has made reference to the possible dangers, but his examples do not seem to be convincing. Generally, structures are more protected than he has assumed. The writer agrees that shrinkage stresses in concrete frames are of definite significance, but, unfortunately, little is known of the extent and magnitude of these colloidal changes, as measured by prevailing moisture conditions in their relation to the impermeability of the mass. Fortunately, these changes are of small significance in temperate climates; but in the tropics they are very destructive.†

The writer has made a general study of this subject, but there is a great deal of specific investigation yet to be done along these lines, before colloidal stresses may be definitely predetermined. As the shrinkage stresses, in their nature, are colloidal, Mr. Way is due special thanks for pointing out their significance. Greater danger, however, is to be found in longitudinal changes

* *Proceedings, Am. Soc. C. E.*, October, 1928, Papers and Discussions, p. 2373.

† *Journal, Inst. of Engrs. (India)*, April, 1925, p. 52.

of beams and girders than in columns, as noted; this is on account of their relative positions and also because setting stresses in columns are confined to one story at a time, and are not cumulative.

From the viewpoint of bringing out the necessity of further investigations along these lines, the discussion has been especially fruitful. Some discussors have wrongly assumed that the paper was intended as a guide solely for concrete work. As a matter of necessity, the concrete designers were long ago compelled to pay attention to continuity in a frame, but steel designers also soon had to "follow suit." To assume a free connection for a girder in computing stresses due to load—and that the same connection was elastic for wind effect—was indeed absurd.

Professor Morris' contribution* is, therefore, especially welcome. He has expressed the desirability for a clear statement of fundamentals and for reserving for special discussion the peculiarities of individual materials. Some discussion has been unnecessarily critical for want of clarity on this point; on the other hand, there have been most excellent supplements with many useful methods, formulas, tables, etc., added to the original paper. The discussions of Messrs. Hyde,† Wilder,‡ Hickerson,§ and Caughey|| are very valuable from this point of view.

Messrs. Wessman, Way, and Snethlage seem to agree that the paper has been unduly critical in regard to conventional practice. Mr. Wessman's¶ experience evidently is limited to more recent years, during which better design methods have been making remarkable progress. His connections, likewise, have been fortunate.

The writer's experience, of being forced to battle against antiquated methods, has been duplicated by many. He has heard statements from progressive and prominent men of experience, that younger engineers have turned cynics because they have often been forced by older men in authority—the commercially minded or the "stick in the mud" variety—to do what they knew to be incorrect.

Mr. Way** seems to have been fortunately placed. His cheerful statement is, therefore, gratifying, but it does not cover the full field. That so many, who are consistently at fault, have failed to offer any contributions is significant as contrasted with so much constructive discussion.

The discussion by Mr. Snethlage†† has special interest, because he admits that there is a "true interpretation of the use of the conventional moment factors" for variable spans, whereas no such use was ever intended by the members of the Joint Committee on Specifications for Concrete and Reinforced Concrete‡‡ or bodies of equal authority. There could be no better demonstration that confused ideas prevail and that greater clarity is needed.

* *Proceedings, Am. Soc. C. E.*, August, 1928, Papers and Discussions, p. 1961.

† *Loc. cit.*, p. 1962.

‡ *Loc. cit.*, p. 1969.

§ *Loc. cit.*, January, 1929, Papers and Discussions, p. 157.

|| *Loc. cit.*, February, 1929, Papers and Discussions, p. 473.

¶ *Loc. cit.*, August, 1928, Papers and Discussions, p. 1964.

** *Loc. cit.*, October, 1928, Papers and Discussions, p. 2373.

†† *Loc. cit.*, February, 1929, Papers and Discussions, p. 483.

‡‡ *Loc. cit.*, October, 1924, Papers and Discussions, p. 1191.

His short method, however, is among the best; but, unfortunately, it is very far from being generally used. It suffers also from an element of uncertainty in that the designer never knows how far from danger he really is, or how much material is being used wastefully.

Professor Mylrea* makes reference to the excellent graphical methods of Messrs. Nishkian and Steinman.† There seems to be one error in the presentation, which must be guarded against in use—the necessity for transforming load values when spans are changed, to compensate for variability of section.

There is an especially clear statement of the economic significance of the writer's formulas and tables from Mr. Hadley.‡ He dwells again on the importance of repeated typical designs, and in a much clearer manner than the writer did in the paper. Possibly several hundred similar beams and joists may be built. Error and waste are, therefore, both multiplied; but, likewise, the economies are worth while when the calculation is done carefully and correctly.

There must be some inducement, however, to designing even the numerous small structures correctly, and no inducements could be more attractive than simplicity of method, accuracy of results, and economy of materials. Therein lies the justification for this paper. It is to be hoped, however, that the tables may soon be supplemented and ultimately supplanted by experimental moment factors obtained by Professor Beggs' method.

Mr. Hadley refers§ to the writer's statement about equi-distant concentrated loads and demands a more complete explanation. Reference is obviously first and foremost to the many so-called joist designs, where complete uniformity may be assumed. It should be noted, however, that even quarter and half-point loading will increase the moments only a very small fraction. Hence, the tables may be used—judiciously, of course—even for concentrated loads within fair limits.

Thanks to Professor Hickerson|| it has been possible to verify again the accuracy of the numerical factors in Table 3.¶ As a result two of these coefficients have been corrected by one unit in the third place. At the same time an added confidence in the reliability of these factors is afforded.

The Joint Committee on Specifications for Concrete and Reinforced Concrete has found a defender in Mr. Way.** The excellent work by the Committee cannot be questioned. It was merely its attitude of inconsistency in regard to proposed moment factors which was deplored. Either the subject should have been treated more fully, or it should have been treated in general terms only. To select the simplest problems and to treat these in great detail, while at the same time omitting completely all the more difficult ones, can only confuse those who specially need the guidance of the report. Because of

* *Proceedings*, Am. Soc. C. E., February, 1929, Papers and Discussions, p. 484.

† *Transactions*, Am. Soc. C. E., Vol. 90 (1927), p. 1.

‡ *Proceedings*, Am. Soc. C. E., November, 1928, Papers and Discussions, p. 2559.

§ *Loc. cit.*, p. 2560.

|| *Loc. cit.*, January, 1929, Papers and Discussions, p. 157.

¶ *Loc. cit.*, March, 1928, Papers and Discussions, p. 719.

** *Loc. cit.*, October, 1928, Papers and Discussions, p. 2373.

frequent errors of design due to misinterpretation of the moment factors specified by the Joint Committee, this criticism seems pertinent.

Erroneous application of conventional moment factors to a series of spans of different lengths, is common—almost universal. It is Mr. Way's belief that the impossibility of creating "an accurate design" has made it necessary for Code Committees to adopt empirical factors. The effect of these empirical factors seems to have been the reverse of what was intended, namely, to induce unqualified people to undertake difficult designs by the simple expedient of letting these very limited factors to serious work in the dark hinterland beyond the borders of their own ignorance.

This obviously raises the question as to the true function, as well as the limitations of a legal "code" or a "committee report" (which, in advance of full knowledge on a subject, anticipates the code). It is not the writer's intention to dwell largely on this, but simply to raise again the point that no committee should endeavor to establish by code or rule simple matters that pertain to gravity, or anything experimentally determinable therefrom. A code should cover only matters of judgment where there is a chance of judicious selection and where the mature judgment of especially selected men will benefit the community as a whole.

MEMOIRS OF DECEASED MEMBERS

FREDERIC VAUGHAN ABBOT, M. Am. Soc. C. E.*

DIED SEPTEMBER 26, 1928.

Frederic Vaughan Abbot was born in Cambridge, Mass., on March 4, 1858. He was a son of Henry L. and Mary Susan (Everett) Abbot, his father having been a Brigadier-General in the United States Army, and an officer of distinction both as a Civil and a Military Engineer.

Mr. Abbot was educated at private schools until he received an appointment to the United States Military Academy at West Point, N. Y., to which institution he was admitted as a Cadet on July 1, 1875. He was graduated in 1879, at the head of his class, and was commissioned a Second Lieutenant, Corps of Engineers, U. S. Army.

He was promoted to the grade of First Lieutenant on June 17, 1881; Captain, on July 22, 1888; Major, on July 5, 1898; Lieutenant-Colonel, on September 9, 1906; Colonel, on June 24, 1909; and Brigadier-General, National Army, on August 5, 1917. He was placed on the retired list on account of physical disability incurred in the line of duty on May 10, 1920, after more than forty years of commissioned service, all of which was with the Corps of Engineers, U. S. Army.

General Abbot had a varied and extensive career of service both as a Civil and as a Military Engineer. After graduating from West Point he was assigned to duty at the United States Engineer School of Application, Fort Totten, New York Harbor. While attending this school he translated and published for the instruction of student officers, Colonel Schiednagel's book on "Submarine Mines" (Spanish), and Schlicting's "Navigable Non-Tidal Rivers" (German). From June, 1882, to August, 1884, he served as Assistant on the improvement of the Mississippi River from St. Louis, Mo., to Cairo, Ill., and on detached service, surveying a portion of the boundary line between Maryland and Virginia. In August, 1884, he became Assistant on the improvement of rivers and harbors in North Carolina and South Carolina, and on the construction of fortifications along the South Atlantic Coast. From May, 1888, to September, 1897, he was in charge of these works, completing the project for obtaining a channel of 21 ft. across the bar at the entrance to the Charleston, S. C. Harbor, by means of converging jetties; constructing a battery for sixteen 12-in. mortars on a swamp foundation; and constructing emplacements for 10-in. guns operating on disappearing gun carriages.

From September, 1897, to August, 1900, General Abbot (then a Major), was in charge of the river and harbor works of Minnesota and Wisconsin, constructing two large reservoir dams at the head-waters of the Mississippi River, and a lock and dam in that river above St. Paul, Minn.

From August, 1900, to April, 1910, he served as Assistant to the Chief of Engineers, U. S. Army, with station at Washington, D. C. In April, 1910,

* Memoir prepared by C. McD. Townsend, M. Am. Soc. C. E.

he was placed in charge of the Boston, Mass., Engineer District, and from June, 1913, to October, 1917, he served as Division Engineer of the Northeast Division Engineer Department, and was in charge of river and harbor construction in portions of New York and New Jersey.

From October, 1917, until his retirement, he again served as Assistant to the Chief of Engineers, acting as Chief of the Division of Operations, Office of the Chief of Engineers, where he supervised the enlistment, organization, training, and forwarding of more than 300 000 engineer troops for service in the World War. He also acted as Chief of Engineers on numerous occasions. During this service General Abbot was a member of numerous boards convened by the War Department or the Chief of Engineers, of which the most important were the Board of Engineers for Fortifications, the Board of Engineers for Rivers and Harbors, the New York Harbor Line Board, the Boston Harbor Line Board, the Joint Artillery and Engineer Board to locate fire-control stations at all seacoast defenses in the United States, and a Board appointed to co-ordinate the work of the Corps of Engineers, Ordinance Department, and Signal Corps.

While General Abbot's services in the field as a Civil Engineer were highly creditable, he obtained his greatest reputation as a Military Engineer and as an Administrative Officer, having been awarded a Distinguished Service Medal by the War Department for his service during the World War, the citation reading as follows:

"For exceptionally meritorious and distinguished service in the organization of Engineer troops and the procurement of enlisted men for the service in the War. His zeal was untiring and the success of his efforts marked."

The closing sentence of this citation can be applied to his entire career, "His zeal was untiring and the success of his efforts marked."

He died on September 26, 1928, and is survived by his widow, *née* S. Julia Dehon, two daughters, Marion B. and Elinor R., and a son, Henry Dehon Abbot.

General Abbot was elected a Member of the American Society of Civil Engineers on December 3, 1884.

ROBERT PAUL ANDERSON, M. Am. Soc. C. E.*

DIED JANUARY 24, 1929.

Robert Paul Anderson, the eldest son of the Rev. R. E. and Alice B. Anderson, was born in Redwood Falls, Minn., on December 11, 1884.

Mr. Anderson received his preparatory education in the grade schools of Heyworth, Ill., the Chambersburg Academy, Chambersburg, Pa., and Grand Prairie Seminary, Onarga, Ill. In the fall of 1906 he entered the University of Illinois, Urbana, Ill., where he spent three years studying Civil Engineering.

After leaving the University Mr. Anderson entered the employ of the Illinois Central Railroad Company and was engaged in the construction of

* Memoir prepared by W. E. Stockwell, Esq., El Paso, Tex.

docks in New Orleans, La., and railroads in Mississippi. Later, he spent a year with the United States River and Harbor Commission on Mississippi River work, and from there he went to the Spokane, Portland, and Seattle Railway Company as Resident Engineer in charge of heavy construction along the Snake River.

In 1909, Mr. Anderson was appointed Locating Engineer for the Northern Pacific Railway Company, working in Montana and Washington. He then became Assistant Engineer on Construction, the youngest man to occupy this position in the entire System.

In 1912, he became Locating and Reconnaissance Engineer for the Farquar Syndicate in South America and three times crossed the Andes in charge of survey parties on important and dangerous work. He had many adventures in little known parts of Brazil, Bolivia, and Chile.

On returning to the United States in 1914, Mr. Anderson was engaged in construction work in El Paso, Tex., and Juarez, Mexico, until the outbreak of the World War, when he enlisted and served in France as Captain of Company E, 55th Engineers, until he was mustered out in July, 1919. During his service in France he was critically ill with pneumonia. This undoubtedly weakened his constitution and contributed to his early death.

Following the war he returned to El Paso and was engaged in work for the City Engineering Department, a notable accomplishment being the construction of the Scenic Drive on Mount Franklin. In 1923, he was appointed City Engineer and served for four years, during which time he had charge of the design and construction of a high-line reservoir of 2 300 000 gal. capacity, with a reinforced concrete dome 140 ft. in diameter, reported to be the largest flat dome in the country. Important street improvements, river protection, and sewerage projects were also carried out during this term of service.

In 1927, Mr. Anderson returned to the contracting business and was Superintendent of Construction for the Ware Company of El Paso until the beginning of his last illness.

Mr. Anderson had a winning personality and is mourned by a host of friends from among all those who came in contact with him.

In 1919, he was married to Mary Emma Jackson, of Paragould, Ark. He is survived by his widow and one daughter, Mary.

Mr. Anderson was elected a Member of the American Society of Civil Engineers on January 18, 1926.

WALTER EUGENE ANGIER, M. Am. Soc. C. E.*

DIED DECEMBER 29, 1928.

Walter Eugene Angier was born at Fitzwilliam, N. H., on May 18, 1863. His father, Philip Doddridge Angier, a native of Fitzwilliam, represented his district in the State Legislature for two terms. His mother, Sarah Arabella

* Memoir prepared by a Committee of the Illinois Section, consisting of E. E. R. Tratman, Assoc. M. Am. Soc. C. E., Chairman, Charles L. Strobel, and Arthur L. Webster, Members, Am. Soc. C. E.

(Reed) Angier, a sister of the mother of the poet, Eugene Field, was the "Aunt Belle" to whom Mr. Field dedicated his "Love Songs of Childhood", and it was at his request that her son was given his name.

The family moved to West Swanzey, N. H., when Mr. Angier was very young, and it was in that town that he was reared, receiving his early education in the common schools and at Mt. Caesar Academy. In 1885, he received his Bachelor of Science degree from the New Hampshire College of Agriculture and Mechanic Arts; and, in 1887, he received the degree of Civil Engineer at the Thayer School of Civil Engineering, Dartmouth College, Dartmouth, N. H.

His first professional experience, during the summer vacation of 1886, was in the office of the late John W. Ellis, M. Am. Soc. C. E., Chief Engineer of the Providence and Worcester Railroad Company, and also with Mr. Dana C. Barber, of Philadelphia, Pa., whom he assisted in surveys and plans for the sewerage system of Lakewood, N. J.

Immediately after his graduation, in April, 1887, he went to Fort Madison, Iowa, as Rodman and Assistant on the Mississippi River Bridge which was being built by the Atchison, Topeka and Santa Fé Railroad Company, under the late Octave Chanute, Past-President, Am. Soc. C. E., as Chief Engineer. In this position, he inspected cement, assisted in the field work, and in the sinking and sealing of the caissons. From October, 1887, to October, 1888, and again in January and February, 1889, he was Sub-Inspector on the harbor improvement at Galveston, Tex., and Recorder on a survey of the harbor and Lower Bay at Galveston. In the interim, from October, 1888, to January, 1889, as United States Assistant Engineer, he was engaged in an examination of the Rio Grande River, from Embudo, N. Mex., to El Paso, Tex., in order to determine its facilities for navigation.

Bridge work was Mr. Angier's particular desire and he left the Federal Service in February, 1889, to go to Memphis, Tenn., as Assistant Engineer on the Mississippi River Bridge, under the late George S. Morison, Past-President, Am. Soc. C. E., as Chief Engineer, and the late Alfred Noble, Past-President, Am. Soc. C. E., as Resident Engineer. He had special charge of all masonry work and the erection of the west approach viaduct. After the completion of the bridge in the summer of 1892, Mr. Morison put him in charge of the construction of a Memorial Library at Peterborough, N. H. For a short time, also, he was engaged on sewer work at Rochester, N. Y.

In November, 1892, Mr. Angier entered the employ of the Illinois Central Railroad Company as Inspector of Bridges, on both shop and field work, and, later, as Assistant Engineer of Bridges, under the late H. W. Parkhurst, M. Am. Soc. C. E., then Bridge Engineer of that Company. He remained there until January, 1902, when he was appointed Resident Engineer on the Thebes Bridge, his third Mississippi River Bridge, under Mr. Noble and Ralph Modjeski, M. Am. Soc. C. E., Chief Engineers. On the completion of the Thebes Bridge, in 1907, he became associated with Mr. Modjeski as his Principal Assistant and, later, was his partner under the firm name of Modjeski and Angier. During the next twenty years he was engaged in the design and erection of several important bridges, his specialty being in con-

nection with difficult foundation work and heavy masonry. In writing of Mr. Angier, Mr. Modjeski states:

"I had a great appreciation for his ability and conscientiousness, which I considered invaluable. His judgment was always characterized by great directness, which often simplified very complex questions. He was of an extremely unassuming nature and a man of very few words."

Mr. Angier suffered a partial paralysis from cerebral lesion in May, 1926, which caused him to retire, although he remained active and retained his interest in engineering work. He was stricken again, at his home in Wheaton, Ill., on Sunday, December 23, 1928, and died on December 29.

In December, 1889, he was married to Mary Powell, of Luling, Tex., who survives him. He leaves also a daughter, M. Estelle Angier, and two sons, Philip Powell Angier, an engineer, of Melville, La., and Robert M. Angier, of Los Angeles, Calif. He was a member of the Western Society of Engineers, and the Chicago Engineers Club.

Mr. Angier was elected an Associate Member of the American Society of Civil Engineers on September 7, 1892, and a Member on September 3, 1902.

LELAND RELLA BALCH, M. Am. Soc. C. E.*

DIED APRIL 18, 1928.

Leland Rella Balch was born in Neillsville, Wis., on March 10, 1883. He was the son of the late Rella W. Balch and Nettie B. (Whipple) Balch, and a descendant in the tenth generation of John Balch, of Somerset, England, who settled in Beverly, Mass., in 1626.

Mr. Balch spent his early life in Neillsville, where he fitted himself for college. He was graduated from the University of Wisconsin with the degree of Bachelor of Science in Civil Engineering in 1905, and received the professional degree of Civil Engineer in 1909.

He spent the summers of 1902-04 as an Assistant with the County Surveyor of Clark County, Wisconsin, and began his professional work with the United States Reclamation Service, first as Engineer's Assistant, then Instrumentman, and, later, as Chief of Party and Resident Engineer in charge of surveys on the Huntley, Sun River, Blackfeet, and Flathead Irrigation Projects in Montana.

From September, 1909, to March, 1910, he took post-graduate work at the University of Wisconsin. In April, 1910, he was appointed an Assistant Engineer with the U. S. Reclamation Service on the Shoshone Project in Wyoming. From November, 1911, to August, 1912, he was engaged as Research Assistant in Hydraulics, at the University of Wisconsin, and spent his time in experimental research and in the preparation of technical *Bulletins* for publication. He prepared several important *Bulletins*, among which may be mentioned: "Tests of Flash Wheels"; "Flow Through Submerged Orifices and Tubes"; and "Hydraulic Curve Resistances". He rendered important service in editorial work in the preparation and revision of various textbooks by D. W.

* Memoir prepared by Daniel W. Mead, M. Am. Soc. C. E.

Mead, M. Am. Soc. C. E., including "Water Power Engineering", "Hydrology", and "Contracts, Specifications, and Engineering Relations".

From 1912 until his death, Mr. Balch was with the firm of Daniel W. Mead and Charles V. Seastone, Members, Am. Soc. C. E., Consulting Engineers, of Madison, Wis. With this firm he had responsible charge of the design and construction of many important works, among which may be mentioned the reconstruction of the municipal pumping plants at Madison, Rockford, Ill., Dubuque, Iowa, and Harvard, Ill.; additions to the power plants of the Madison Gas and Electric Company, at Madison; the Mississippi Valley Public Service Company, at Winona, Minn.; the Peninsular Power Company, at Iron River, Mich.; the Eastern Oregon Light and Power Company, at Baker, Ore.; and of the design and construction of the Diesel electric plant of the Peninsular Power Company, at Iron River.

He also participated in investigations and preparation of reports; in the design of numerous water-works, water power, and sanitary works; in divers appraisals and valuations; and also in numerous investigations and reports on power projects and flood-protection works.

Mr. Balch was a man of high ideals, of sterling character, of the highest integrity, and of marked ability. Throughout his career he adhered to the highest of moral and professional standards. A man of pleasing personality, he possessed a genial disposition, and a keen sense of humor, endearing himself to all who knew him.

Mr. Balch was married in Chicago, Ill., on June 15, 1918, to Margaret Elizabeth O'Reilly. Their children are Margaret O'Reilly, Dorothy Jane, James Leland, and Ruth O'Reilly.

He was a member of the American Society of Mechanical Engineers, and the Engineering Society of Wisconsin; an Associate Member of the Wisconsin Utilities Association, and a Member of the Madison Technical Club. He was also a Thirty-second Degree Mason and a member of the Blackhawk Country Club of Madison.

Mr. Balch was elected an Associate Member of the American Society of Civil Engineers on October 5, 1909, and a Member on August 12, 1920.

GEORGE STRONG BAXTER, M. Am. Soc. C. E.*

DIED JULY 2, 1928.

George Strong Baxter was born in New York, N. Y. on November 21, 1845, the son of George and Anna (Strong) Baxter. His childhood and school days were spent mainly in New York, after which he went to Williams College at Williamstown, Mass., from which he was graduated in 1865. Three years later, he was graduated from the Columbia University School of Engineering.

From 1868 to 1872, Mr. Baxter was employed in land surveying in various parts of the United States. In New York City, he constructed the foundations for the Second and Third Avenue Elevated Lines. He was engaged in the

* Memoir prepared by Mrs. E. Vogelsang, New York, N. Y.

Fourth Avenue Improvement in 1872, became Assistant Engineer, and built the New York Central Viaduct on Park Avenue, north of 96th Street.

In 1879, he became Cashier for the Northern Pacific Railroad Company, and, in 1885, was appointed Assistant Treasurer and sent to St. Paul, Minn. In 1888, he rose to the office of Treasurer and returned to New York. Later, when the Company went into the hands of Receivers, he remained as Treasurer until it was discharged from the Receivership.

In 1895, Mr. Baxter founded the cross-tie business of G. S. Baxter and Company, of New York and Jacksonville, Fla. At first, the firm bought railroad ties in the South and shipped them to New York, but, later, the Company expanded and bought lands in Georgia where it cut its own ties; one saw-mill was constructed in Fargo, Ga. In 1900, the Company handled more Southern Pine cross-ties than all other dealers combined.

Owing to some trouble with the Atlantic Coast Line Railroad Company, over shipping ties to tide-water, the Company built a short line east to connect with the Southern Railroad. Mr. Baxter then conceived the idea of a railroad line east to Jacksonville, and west to Valdosta Ga. The Valdosta Terminus made a connection with the Middle West cities and a short route from those cities to the Florida East Coast. The Atlantic, Valdosta and Western Railroad Company was then incorporated with Mr. Baxter as Chairman of the Executive Committee. The various railroad companies fought hard to prevent the admission of the Atlantic, Valdosta and Western Lines to the Union Station in Jacksonville, but they were finally defeated and the railroad was admitted to the Union terminal rights. Later, the Atlantic, Valdosta and Western Railroad Company considered building west from Valdosta to Birmingham, Ala., and to prevent this, the Southern Railroad Company finally bought the line.

Mr. Baxter was greatly interested in the creosoting of cross-ties, and at one time considered establishing a plant for that purpose. He was also interested in the possibility of bringing tropical hard wood cross-ties into the United States. As the World War cut into the railroad tie business, G. S. Baxter and Company established a shipyard in Jacksonville, on which Mr. Baxter was engaged until 1918 when the partnership was dissolved, and he retired.

In 1874, he was married to Emmeline Carnes Weeks, of New York, the daughter of Emmeline Carnes and Edward Augustus Weeks. She died in 1921. They had five children, of whom three survive: Wyllys Pomeroy Baxter of New York City, George Strong Baxter, Jr., of Westerly, R. I., and Mrs. Erwin Vogelsang, of New York City.

Mr. Baxter was a member of the University and the Mid-Day Clubs. After his return from St. Paul, he resided in New York, and had a summer home at Bellport, L. I. A man of simple tastes and strong affections, he was a most devoted husband and father. His was an unusual intellect and, to the time of his death, he retained his grasp of affairs and interest in life.

Mr. Baxter was elected a Junior of the American Society of Civil Engineers on May 12, 1875, and a Member on May 3, 1876.

WILLIAM HERBERT BIXBY, M. Am. Soc. C. E.*

DIED SEPTEMBER 29, 1928.

William Herbert Bixby, the son of Clark Smith and Elizabeth (Clark) Bixby, was born in Charlestown, Mass., on December 27, 1849. He was descended from Colonial stock, his forebears having served in the Colonial and Revolutionary Wars. He attended the public schools of Brookline and Cambridge, Mass., and then entered the Massachusetts Institute of Technology. After one year at the latter institution he was admitted in 1869 to the United States Military Academy at West Point, N. Y., from which he was graduated in 1873, at the head of his class, and commissioned a Second Lieutenant in the Corps of Engineers, U. S. Army.

Lieutenant Bixby's first assignment was at the Engineer School of Application, Willets Point (now Fort Totten), N. Y., from 1873 to 1875. In 1875 he was assigned to duty at the United States Military Academy where he served as Instructor and Assistant Professor of Civil and Military Engineering until 1879. In 1879 he was sent on duty to Europe where he remained until 1882, attending the Ecole Nationale des Ponts et Chaussées of France, from which he was graduated with honor in 1881. During his tour of duty abroad, he also made numerous investigations of European practice in civil and military engineering. He received the decoration of Chevalier of the Legion of Honor of France for assisting at the maneuvers of the French Army in 1880.

Returning to the United States, he was placed on duty with the Engineer Battalion at Willets Point, where he remained until 1884. By this time he had reached the grade of Captain in the Corps of Engineers, and, from 1884 to 1910, he served as Captain, Major, Lieutenant-Colonel, and Colonel, on river and harbor or lighthouse duty at Wilmington, N. C., Newport, R. I., and Philadelphia, Pa.; at Cincinnati, Ohio, in charge of the Ohio River System; on the Great Lakes at Detroit, Mich., and Chicago, Ill.; and at St. Louis, Mo., as President of the Mississippi River Commission. In 1910, he was appointed Brigadier-General, Chief of Engineers, and served in this capacity until his retirement from active service in 1913.

On the entrance of the United States into the World War in 1917, General Bixby was recalled to active duty, and served until 1919 supervising the work of river and harbor improvement on the Mississippi River and its tributaries, in order to release officers on the active list for war service.

The foregoing outline of General Bixby's service gives only a general indication of the character and variety of his duties during his long and active life. In addition, he was a member of many Official Boards, which greatly extended the scope of his activities; and it is no exaggeration to state that there are few of the manifold activities of the Corps of Engineers to which he did not contribute during his active service of more than forty years. Possessed of a brilliant mind, a tireless energy, and a seemingly unlimited

* Memoir prepared by Herbert Deakyne, Brig.-Gen., U. S. Army, M. Am. Soc. C. E.

capacity for study and research, he acquired an almost inexhaustible fund of knowledge of civil engineering works, on which he drew in the consideration of the multifarious problems that presented themselves during his career.

One important board of which General Bixby was a member was appointed by the Chief of Engineers in January, 1894, pursuant to instructions of the Secretary of War, the late Hon. Daniel S. Lamont, to determine the maximum practicable span for suspension bridges. The other members were the late Gen. (then Major) Charles W. Raymond, U. S. Army, M. Am. Soc. C. E., and Col. (then First Lieutenant, Corps of Engineers) Edward Burr, U. S. Army, (*Retired*), M. Am. Soc. C. E. The appointment of the Board was due to strenuous efforts made by interested parties to secure the passage by Congress of a bill authorizing the construction of a bridge across the Hudson River at New York, N. Y., and specifically authorizing a pier about mid-channel. The proponents of the bridge maintained that it was a crying necessity and could not be built as a single span or without a channel pier. The Chief of Engineers strongly opposed the deliberate placing of any such obstruction as a bridge pier in mid-channel. Mr. Lamont directed the Chief of Engineers to convene a Board of Officers to investigate and report its conclusions as to the maximum length of span practicable for suspension bridges and consistent with an amount of traffic probably sufficient to warrant the expense of construction.

The members of the Board, as is customary in such cases, were given this work in addition to their regular duties. General (then Captain) Bixby, at this time, was on duty in charge of the U. S. Engineer Office at Newport, R. I. The Board, after a study of the task before it, assigned to each member particular lines of investigation, in addition to a general study of the problem as a whole. Among other things, the subject of wind pressures was assigned to Captain Bixby. The result of his investigations is printed as an Appendix to the Board's report and is characteristic of his efficiency, thoroughness, and almost infinite capacity for detail. Captain Bixby's compilation and discussion was unique in its field and filled a need long felt by engineers of that day, particularly in bridge design. The report of the Board was printed not only as an official document but also in the engineering press. *Engineering News*, in addition to printing the report, stated in an editorial* that the report "forms one of the most valuable and instructive engineering investigations of the day".

The report set the accepted possibilities as to length of bridge spans far in advance of the practice then current, and preserved the Hudson River and doubtless many other important waterways from serious obstruction by bridge piers. The conclusions of the Board are being justified by the construction, now under way, of a bridge across the Hudson with a span of 3 500 ft., which is still well within the maximum given by the Board of about 4 300 ft.

In 1909, General (then Colonel) Bixby was selected as Advisory Engineer to the National Waterways Commission, a body created by the Act of Con-

* *Engineering News*, November 22, 1894.

gress of March 2, 1909, which Commission he accompanied to Europe. His detailed knowledge of works for the improvement of navigation both in Europe and the United States made him of great value to the Commission.

The wreck of the battleship *Maine* was removed from Havana Harbor while General Bixby was Chief of Engineers. He was deeply interested in this work, and gave it a great deal of personal attention, until the success of the undertaking was assured.

General Bixby was as active physically as mentally. As a young man he was an expert skater and, when on duty in France, took occasion during one winter to visit Holland in the hope of acquiring some of the refinements in the art. It developed, however, that his ability as a skater was already so great that he became a teacher rather than a student of the Hollanders. He found time while abroad for mountain climbing and ascended the Matterhorn with two guides. On another occasion he left Zermatt, Switzerland, about noon with one guide and ascended the Breithorn. Returning to the village about noon the next day he met a party starting out to make the ascent. They urged him so strongly to accompany them that he consented and re-ascended this mountain, getting back to the village about 4:00 P. M. of the following day, after more than 48 hours of continuous mountain climbing.

General Bixby was always intensely interested in new devices. As an Instructor at the United States Military Academy, during the period from 1875 to 1879, he foresaw the possibilities of the typewriter, then a crude machine compared to present-day models, and endeavored to secure its introduction to take the place of the laborious and time-consuming pen and ink method by which Army correspondence was then conducted.

He also took a great interest in the improvement and simplification of administrative procedure and was largely responsible for a number of methods now in use by the Corps of Engineers. One of these was the standard specifications for dredging, which he initiated and which are now in use throughout the Engineer Department for all dredging contracts. His interest extended to almost every field of scientific endeavor. The number of scientific publications to which he subscribed was so great that it was his custom to rent a private office to which all such periodicals were sent and where his professional library was kept in order to avoid overtaxing the facilities of the office assigned him for the transaction of public business.

General Bixby was a man of simple tastes; there was nothing of the martinet about him. He loved the discussion of matters of professional interest, and his tremendous fund of knowledge on military and civil engineering, and other scientific subjects, was a source of astonishment to those to whom it was displayed. He was one of the most generous and kind-hearted of men, always thoughtful of the feelings and comfort of others, and most considerate in his dealings with them. His character was unimpeachable, and he leaves a record of service to his country unsullied by the slightest spot of selfish interest.

He belongs to that remarkable group of engineers, now nearly all gone, whose active lives spanned the period between the close of the Civil War and

the outbreak of the World War; an era in which the advance in scientific knowledge and engineering achievement was so great, that some future historian of scientific progress may well describe it as the "Golden Age" of Science.

Deeply interested in scientific progress throughout his active life, General Bixby's interest in science and engineering continued undiminished to the end. In spite of advancing years, he was vigorous both in mind and body until stricken with his last illness when death stopped forever the strivings of his restless intelligence. He died in Washington, D. C., and after a simple service at his home, his ashes were placed in the Arlington National Cemetery.

He was married on December 27, 1893, to Mrs. H. M. Jones, of Philadelphia, the widow of Lieut. H. M. Jones, 4th U. S. Artillery. Mrs. Bixby survives him.

General Bixby was a member of the French Society of Civil Engineers, the British Institution of Civil Engineers, the American Society of Mechanical Engineers, the Institute of Construction Engineers, the Academy of Political and Social Science, and other societies of Political Economy, the American Association of Port Authorities, and the International Association of Navigation Congresses. He was President of the International Congress of Navigation, held at Philadelphia in 1912; Chairman of the Engineering Section of the Pan-American Scientific Congress at Washington, D. C., in 1915; President of the American Society for Testing Materials in 1917-18; and President, in 1918-19, of the American Society of Terminal Engineers.

He also held memberships in the Society of American Military Engineers, the U. S. Military Service Institution, the U. S. Infantry Association, the U. S. Cavalry Association, the U. S. Naval Institute, the American and British Associations for the Advancement of Science, the Mathematical Society of America, and the American Academy of Arts and Sciences.

General Bixby was elected a Member of the American Society of Civil Engineers on April 5, 1882.

EUGENE DICKINSON BURNELL, M. Am. Soc. C. E.*

DIED FEBRUARY 20, 1928.

Eugene Dickinson Burnell was born at Steilacoom, Wash., on December 11, 1884, the son of Dr. A. T. and Mary (Frayer) Burnell.

Mr. Burnell attended Cornell University, at Ithaca, N. Y., from which he was graduated with the degree of Civil Engineer in 1906. From July, 1906, to December, 1908, he was employed on drafting and estimating by Hugh L. Cooper, M. Am. Soc. C. E., Consulting Engineer, of New York, N. Y., and, later, as Foreman on hydro-electric power plant construction, including the design of hydro-electric power plants.

He left the employ of Mr. Cooper in January, 1909, to fill an engagement as Assistant Engineer for the City of Pensacola, Fla.; he did not remain

* Memoir prepared by Joseph D. Evans, Esq., New York, N. Y.

long in this work, however, because of his appointment to a position, in which greater experience could be obtained, with the Isthmian Canal Commission on the construction of the Panama Canal. He went to Panama in February, 1909, remaining there until January, 1910, as Assistant Engineer in the Department of Masonry Design, being particularly concerned with the design of the locks, gates, and valves, at Culebra.

In February, 1910, Mr. Burnell accepted a position with the J. G. White Engineering Company, of New York. At first, his duties included reporting on proposed hydro-electric developments, designs, and estimates. Later, he was appointed Assistant Superintendent of Construction on the 30 000 h.p. hydro-electric plant of the Canadian Light and Power Company, at Montreal, Que., Canada.

At the completion of this work in February, 1913, Mr. Burnell accepted a position as Construction Engineer for the Phoenix Construction Company, at Grace, Idaho, on the construction of the Utah Power and Light Hydro-Electric Development, one of the properties of the Electric Bond and Share Company. On this work he made evident his capabilities as a Cost Analysis Engineer.

Mr. Burnell's work in Idaho was completed in the spring of 1914, and he then accepted a position with the Power Construction Company of Shelburne Falls, Mass., as Construction Superintendent in charge of building ten miles of a standard-gauge railroad. He finished this work in September, 1914. In January, 1915, he was appointed Superintendent of Construction for the Aqueduct Construction Company, Winnipeg, Man., Canada, on the construction of ten miles of a concrete aqueduct, which position he retained until the work was completed in April, 1916.

When Mr. Burnell returned to New York, he was employed by the Agency of the Canadian Car and Foundry Company of New York, as Assistant Works Manager, at its large shell and shrapnel loading plant at Kingsland, N. J. It was during his engagement on this work that he showed his forcefulness as an efficient Production Manager, and it was largely through his technical ability, energy, and painstaking efforts that the Company was able to fulfill the contract on which the production had fallen far behind. The penalties that were being enforced would have meant a great financial loss to the Company. Through Mr. Burnell's efforts, however, work under this contract with the Russian Government for loading 2 500 000 18-lb., high-explosive shells and 2 500 000 rounds of shrapnel, at an aggregate price of \$83 000 000, was brought up to large quantity production and the Company's penalties were soon cancelled by very substantial bonuses, all of which were in excess of the original contract price.

In April, 1917, soon after the United States entered the World War, the Evans Engineering Corporation, of New York, which had been formed for the purpose of loading 18-lb., high-explosive shell for the United States Army, employed Mr. Burnell as Chief Engineer in charge of design and construction of the \$800 000 shell-loading plant which this Company was building at Old Bridge, N. J. Later, he was placed in charge as Works Manager.

Although this plant, which was a privately owned corporation, loading shells on a unit-price basis, with contracts from the United States Ordnance Department, was the smallest of the shell-loading plants in the United States, it holds the record for loading the greatest number of 18-lb. shells (75-mm. high-explosive) in any one day, month, or for the duration of the World War, and this work was carried on under the able direction of Mr. Burnell.

One of the financial interests in the Evans Engineering Corporation caused the Atlantic Loading Company to be formed, for the purpose of constructing an \$18 000 000 to \$25 000 000 high-explosive shell-loading plant for the United States Government. This plant was constructed later and operated at Amatol (Hammonton), N. J. In conjunction with his duties as Works Manager of the Evans Engineering Corporation plant at Old Bridge, Mr. Burnell was appointed Chief Engineer on design for the Amatol plant of the Atlantic Loading Company. When this latter Company's plant was sufficiently constructed to permit operations of loading high-explosive shell, Mr. Burnell devoted his entire time to serving as Works Manager at Amatol. Of the four plants of this type which the Government constructed, this one was reputed to have been the largest. It had a capacity of turning out more than 150 000 high-explosive shells daily, in sizes ranging from 18 lb. (75 mm.) to 80 lb. At the termination of the war, the Government selected this plant for carrying on several years' operations, scrapping the others which had been constructed under agency form of contracts. The Atlantic Loading Company terminated its work in the summer of 1919. Mr. Burnell's work during the war stands out eminently as an accomplishment second to none for its efficiency and harmonious operation.

In 1919 Mr. Burnell joined the Overman Cushion Tire Company, of New York, as Assistant to M. C. Overman, President; later, he was made a Vice-President. As is so marked in his career, he again showed himself to be of a constructive turn of mind and was instrumental in systematizing the entire operations of this Company.

In August, 1917, Mr. Burnell was married to Augusta A. Evans, who survives him. He is survived also by his father and mother and three sisters, Elizabeth Burnell, of Los Angeles, Calif., Mrs. Bernice Maricle, of Everson, Wash., and Mrs. Esther Mills, of Longs Peak, Colo.

All those who knew Mr. Burnell soon learned to appreciate that he was an engineer and business man of ability, one whose loyalty and integrity ever could be counted upon. He was quiet, unassuming, and always had a kind word for every one with whom he came in contact.

He was a member of the Cornell University Club, New York, the Rod and Bob Club of New York, the Rockwood Hall Country Club, the Society of Automotive Engineers, and the American Society of Mechanical Engineers.

The passing of Mr. Burnell leaves a vacancy in the hearts of all his friends, and takes from the world an engineer and operating manager who, had he lived, might have accomplished still greater things in useful construction.

Mr. Burnell was elected an Associate Member of the American Society of Civil Engineers on June 4, 1913, and a Member on September 9, 1919.

DAVID SYLVANUS CARLL, M. Am. Soc. C. E.*

DIED NOVEMBER 5, 1928.

David Sylvanus Carll was born in Huntington, N. Y., on March 21, 1855, the son of David and Mariette (Parsons) Carll. He was graduated from the Huntington Union School in 1875.

Mr. Carll's professional work began immediately after his graduation, his early experience consisting of surveying in the vicinity of his home. From 1877 to 1881, he was Surveyor to various commissioners of highways. Subsequently, he was Transitman and Assistant Engineer for the Long Island Railroad Company, Assistant Engineer for the New York, West Shore, and Buffalo Railway Company, and he was also engaged in practice as a Civil Engineer and Surveyor in Saugerties, N. Y.

From 1884 to 1888, Mr. Carll was with the Department of Public Works of New York City, in connection with the construction of the new aqueduct and reservoirs. In March, 1887, he went to Kansas City, Mo., where he was engaged on the construction of the cable railway lines, his first employment being with the Grand Avenue Railway Company, and, later, as Assistant Engineer with Knight and Bontecou, Consulting Engineers. From October, 1889, to 1890, he was Superintendent of Grading and Construction in the City Engineer's Office in Kansas City.

In November, 1890, Mr. Carll began his career as Engineer Executive for the Washington and Georgetown Railroad Company, now The Capital Traction Company, of Washington, D. C. His first work in Washington was in charge of installing the cable system on the lines of that Company. On the completion of the work in 1892, he was made Chief Engineer and continued in that position, in charge of engineering work, until 1908. Subsequent to 1895, he became the Superintendent of the Company, retaining his position as Chief Engineer, and was responsible for all the Company's Operating as well as its Engineering Departments. During the construction of the cable road his duties included not only the construction of the cable lines themselves, but of a cable power station and office buildings on Pennsylvania Avenue, on the site of the present District Building. He was also engaged on the construction of several car barns and other structures.

After the destruction of the Cable Power Station by fire in September, 1897, the lines of the Company were electrically equipped; this was the first instance of the conversion of cable lines to the underground conduit system. Mr. Carll was in charge of all this work as Chief Engineer, including, in addition to the reconstruction of track, the building and equipment of a direct-current power station, the rebuilding of numerous car barns, and the design and purchase of rolling equipment. This work was done principally during 1897 and 1898, and at this time Mr. Carll's responsibilities also included the operation of all the Company's street railway lines, which for a time included operation by horse cars, cable, over-head trolley, and underground conduit. In

* Memoir prepared by J. H. Hanna, M. Am. Soc. C. E.

February, 1908, Mr. Carll was made a Director of The Capital Traction Company and elected to the office of Vice-President and General Manager. He remained a Director and Vice-President until his death, having given up his more active duties with the Company in 1916.

During Mr. Carll's long residence in Washington he showed great interest in the development of the Capital City by active association with many civic organizations. He had been a prominent member of the Board of Trade for many years and was Vice-Chairman of its Committee on Law and Order. He was a Charter Member, and served on the first Board of Directors of the Washington Society of Engineers; he also served as Vice-President in 1908 and as President in 1909. He had been prominently active in the District of Columbia Section of the American Society of Civil Engineers since its organization and served as Chairman in 1920. He was a member of the Cosmos Club, of Washington, and many other civic and welfare organizations.

The Mt. Pleasant Congregational Church of Washington was, next to Mr. Carll's business and profession, his greatest interest. A member of the church since 1894, he was a Trustee for more than twenty years and Chairman of its Building Committee during the erection of its present structure.

Mr. Carll was married on February 22, 1883, to Maria B. Dawes, who died in 1922, leaving one daughter, Mrs. Ernest D. Roberts, of Washington. In 1925, he was married to Eliza R. Davis, of Auburn, N. Y.

To those who were fortunate enough to be associated with David S. Carll and to have his friendship, his outstanding characteristic was his humanity and love for his fellow man. An able engineer, a forceful, energetic executive, always carrying heavy responsibilities, he never failed to find time for a word of encouragement or more substantial help for any who came to him. The love of the thousands of men and women who had the privilege of working under his direction is the outstanding feature of his successful career—his bequest to them, the memory of a Christian gentleman.

Mr. Carll was elected a Junior of the American Society of Civil Engineers on March 7, 1888; an Associate Member on July 1, 1891; and a Member on October 7, 1896.

WILLIAM GODLEY COUGHLIN, M. Am. Soc. C. E.*

DIED OCTOBER 12, 1927.

William Godley Coughlin, the son of Samuel Britton and Anna Margaret (Weber) Coughlin, was born on February 15, 1862, in Florence, N. J. His father was of English and Irish ancestry, whereas his mother's family came to the United States from Germany in 1833. The boy received his education in the public and private schools of Florence and, later, attended the Farnum School, at Beverly, N. J., preparatory to entering Lehigh University, at Bethlehem, Pa.

* Memoir prepared by W. C. Cushing, M. Am. Soc. C. E.

During this period, his father decided to join the Collins Expedition of 1878, with the purpose of engaging in railroad construction in Brazil. As the prospect of such an adventurous trip to South America appealed strongly to the lad's imagination, he went with the Expedition and obtained the position of Rodman on the Madeira and Mamore Railway around the Falls of the Upper Madeira in Brazil. The Expedition seems to have been ill-fated, for those of the party sailing on the second ship, the *Metropolis*, were lost at sea, Mr. Coughlin having escaped that fate by his persistent petitions to be allowed to accompany his father.

On his return to the United States, after a year and a half of service in South America, Mr. Coughlin became engaged in preliminary surveys for a railway from Pittston to Hawley, Pa. He then entered the service of the Pennsylvania Railroad Company, which was the beginning of a life service in the Engineering, Maintenance of Way and Structures, and Transportation Departments of that System. He was employed, successively, as Rodman, Levelman, and Transitman on the following work: Preliminary surveys and location for a branch line from Baltimore, to Catonsville, Md.; survey for a proposed pleasure railway through Fairmount Park; re-location and grade revision of the Western Pennsylvania Railroad; preliminary surveys and location of the Cornwall and Lebanon Railroad; revision of alignment on the main line of the Pennsylvania Railroad; location and construction of the Philadelphia and Long Branch Railway from Whittings to Bay Head, N. J.; fourth track work on the Western Division of the Pennsylvania Railroad; and location and construction of a connecting line between Dauphin and Rockville, Pa.

In March, 1883, Mr. Coughlin entered the Maintenance of Way Department of the Northern Central Railway (Pennsylvania Railroad) as Assistant Supervisor in the office of the Engineer of the Baltimore Division and, in May, 1886, he was appointed Supervisor of the West Jersey and Seashore Railroad (Pennsylvania Railroad) in charge of maintenance-of-way work, which position he held until March, 1893.

The following include some of the positions which Mr. Coughlin held during the period extending from March, 1893, to April 1, 1907: Assistant Engineer on the Baltimore Division of the Northern Central Railway, in full charge of engineering work; Principal Assistant Engineer on maintenance-of-way work on the Erie Division of the Pennsylvania Railroad, including that territory extending from Erie, Pa., to Baltimore, Md., and numerous lateral branches; and Superintendent in charge of operations of the Elmira Division of the Northern Central Railway, with headquarters at Elmira, N. Y.

In April, 1907, Mr. Coughlin was appointed Superintendent of the Erie Division and, in March, 1909, of the Renovo Division, of the Pennsylvania Railroad Company, in which positions he had charge of operations, including engineering work, maintenance of way, bridges, buildings, construction of piers, and dredging.

From November, 1913, until his retirement in October, 1927, Mr. Coughlin held the positions of Engineer and Chief Engineer of Maintenance of Way

for the Pennsylvania Railroad Company, having his office at Philadelphia, Pa. Just prior to his retirement, he was occupying the position of Special Agent with this Company.

Mr. Coughlin's faithful and loyal performance of duty appeared everywhere in his work. Gifted with an especially analytical mind, he was constantly active in inventions for the improvement of railroad structures and appliances, with a view to economy and improved elements in the management of his territorial range. He was granted several patents by the United States Patent Office, the last being received a few months before his death. These patents were closely allied to railroad construction and demonstrated the possible achievement of an active imagination coupled with a skillfully managed pencil. His work along this line made evident his earnest efforts to improve the track structure under his official jurisdiction and his vigorous activity in these conceptions during his life time.

Mr. Coughlin was of a modest and retiring disposition, possessing withal a very pleasing personality. Many anecdotes are told by his associates to illustrate his ready wit and humorous outlook. His was a bright, cheery, and sympathetic nature.

He was a member of the Engineers Club of Philadelphia and of the Madeira and Mamore Association, and a communicant of the Protestant Episcopal Church.

As Mr. Coughlin was not married, he lived during the last fourteen years of his life, with his family, at the "old home" in Burlington, N. J.

Mr. Coughlin was elected a Member of the American Society of Civil Engineers on March 4, 1913.

ELMORE DAVID CUMMINGS, M. Am. Soc. C. E.*

DIED NOVEMBER 17, 1927.

Elmore David Cummings was born on November 14, 1861, in Indiana, Pa. His parents, Joseph and Margaret Shields Cummings, were farmers, and the early life of their only son was spent entirely on the farm. A natural aptitude for study and ambitions along other than farming pursuits brought about on the part of the youth an early determination for a college education along engineering lines. Aided and assisted in his studies by his parents, Mr. Cummings accomplished practically his entire preparation for college at home, save for a part of a year, at the age of 20, which he spent at Washington and Jefferson College.

Mr. Cummings entered Cornell University at Ithaca, N. Y., in 1885 and was graduated as a Civil Engineer four years later with honorable mention for his graduating thesis. He immediately entered the employ of G. W. G. Ferris and Company, of Pittsburgh, Pa., as Inspector of Materials at mills, shop construction of railway bridges, and structural work. The early and thorough insight thus gained in structural steel shop work, together with the fact that

* Memoir prepared by W. C. Weeks, M. Am. Soc. C. E.

he kept abreast of improvements in shop machinery and methods of fabrication, was of inestimable worth in his life work, which was, for the most part, that of a designer in steel. As he designed, he visualized with rare insight the various shop processes necessary to produce in tangible form the creation of his mind, and the value of his work was greatly enhanced thereby.

From 1891 to 1893, Mr. Cummings was Draftsman and Assistant Designer for the Columbus, Ohio, Bridge Company, and for the late E. L. Corthell, Past-President, Am. Soc. C. E., Consulting Engineer, on large railway bridges. Following these engagements he was an Instructor in Civil Engineering at Washington University, St. Louis, Mo.; an Assistant Engineer to the late J. B. Johnson, M. Am. Soc. C. E., in the laboratory work of the United States Timber Physics Investigations at this same Institution, and after the completion of these investigations, he was again engaged in instructional work, this time as Assistant Professor of Civil Engineering at the University of Maine, Orono, Me. His natural predilection for designing, however, led to his return to that field of endeavor, and from 1896 to 1899 he was Office Engineer and Assistant Designer for the Pittsburgh Bridge Company in charge of contractual relations and design pertaining largely to intricate roof construction and the design of mill buildings.

In 1899, the United States Engineer Office at St. Paul, Minn., engaged Mr. Cummings as Designer and Principal Draftsman under the general supervision of the Engineer Officer for special work of design in connection with the reconstruction of the reservoir dams at the head-waters of the Mississippi River, and the construction of locks and dams between Minneapolis, Minn., and St. Paul. This work demanded many detailed investigations of governing considerations, and was characterized by the most painstaking thoroughness and the exercise of a high degree of originality in design. He was possessed of an unusual ability to convince others that the devices and structures he proposed would function properly when they were fabricated and installed. The signal success of Mr. Cummings' work at this time was an accomplishment which his surviving associates of that period still remember.

From 1901 to 1903, Mr. Cummings was Designing Engineer in the Bridge and Building Department of the Chicago, Milwaukee and St. Paul Railway Company, where his work ranged from the design of culverts, shops, and chimneys, to that of bridges spanning the Mississippi River. In November, 1903, he became Junior Engineer and, later, U. S. Assistant Engineer under the immediate direction of the Chief of Engineers, at Washington, D. C.

Mr. Cummings' work in the office of the Chief of Engineers was particularly concerned with the design, construction, installation, and testing of ammunition hoists and disappearing searchlight towers for sea-coast fortifications. His work demanded special qualifications and an exercise of originality far beyond the capabilities of most designers. His accomplishments in this difficult work were of outstanding merit. He not only made many original studies of highly technical character, but he also developed a considerable number of machines and mechanical structures, and incidentally made numerous inventions on which the Government took out patents in his

name. It is noteworthy that without exception the creations of his genius have proved to be eminently successful for the purpose for which they were designed. Save for one instance, descriptions of these devices, although well meriting publication in the technical press, have never appeared, due to their confidential character. By special permission of the Chief of Engineers, a descriptive paper on disappearing searchlight towers prepared by Mr. Cummings was published in February, 1924, by one of the leading engineering weeklies.

During the World War, Mr. Cummings handled the technical features pertaining to fabricated materials valued at many millions of dollars. This material was sent to France and consisted for the most part of framed structural steel for large warehouses designed for ready erection in the field.

Mr. Cummings' active life was devoted to engineering, largely in the field of engineering design in which he particularly excelled. All his work was characterized by thoroughness, simplicity, and adaptability to the purpose for which it was intended. His creations were the result of the most careful and painstaking effort, and they were notable for pronounced simplicity and practicability.

Mr. Cummings, by nature a deeply religious man, was a life long member of the Presbyterian Church. He was married on August 7, 1895, to Jessie Williams, of Cleveland, Ohio, who, with a son, Carl Elmore Cummings, survives him.

He was a member of the Washington Society of Engineers, the Society of American Military Engineers, and the Cornell Clubs of New York, N. Y., and Washington, D. C.

Mr. Cummings was elected an Associate Member of the American Society of Civil Engineers on November 4, 1903, and a Member on June 30, 1911.

HANS HELLAND, M. Am. Soc. C. E.*

DIED MARCH 9, 1928.

Hans Helland, the son of Hans and Marie Helland, was born at Bergen, Norway, on August 10, 1854. He received his early education at the Primary and High Schools of Bergen and Christiania. He then attended the Technical Institute at Trondhjem, and, still later (1879), was graduated from the Royal Saxon Polytechnicum, of Dresden, Saxony, Germany, with the degree of Ingenieur Civil.

Prior to his graduation in 1879, however, Mr. Helland had served his apprenticeship in the Laxevoag Machinery Factory, and had been an Assistant Engineer on the Royal Norwegian Railways. The year following his graduation he became Statistician for the Royal Railways and the Department of Highways of Norway.

In 1881, he emigrated to America, settling in Texas, where he became imbued with the "spirit of the West", and henceforth remained to devote a

* Memoir prepared by Rufus C. Thaxton, Assoc. M. Am. Soc. C. E.

useful life to transforming a frontier State to its present highly industrialized civilization. The crying need of that day was the construction of railroads, and naturally Mr. Helland assumed his share of this duty. On his arrival in Texas in 1881, he immediately became a Construction Engineer on the Texas Central Railroad and remained with that Company until 1889, when he became Vice-President and General Manager of the Central Texas and Northwestern, and the Fort Worth and New Orleans Railroad Companies. When these two lines were consolidated with the Houston and Texas Central Railroad in 1902, he became Maintenance-of-Way Engineer for the entire System (now a part of the Southern Pacific System). He remained in this capacity until 1906, when he resigned to locate and construct the Panhandle Short Line. In 1908, he became Maintenance-of-Way Engineer for the San Antonio and Aransas Pass Railroad Company, with which he remained until 1913.

From 1913 to 1921, during its first struggle toward greater improvements, the City of San Antonio, Tex., employed him as its City Engineer. It is noteworthy that, after approximately thirty-two years of active building and improvement of railways in Texas, Mr. Helland should then take up the work of enlarging a pioneer city to one of metropolitan proportions.

After 1921 he was engaged in the private practice of engineering in San Antonio, until his death, when his son Hans Helland, III, assumed the direction of the office.

Mr. Helland was a member of the American Society for Municipal Improvements and the San Antonio Scientific Society; he was also a member of the Polyteknisk Forening of Norway, and of the Ingenieur Verein of Dresden, Saxony.

Mr. Helland was elected a Member of the American Society of Civil Engineers on June 3, 1915.

HAROLD VINCENT JOSLIN, M. Am. Soc. C. E.*

DIED NOVEMBER 3, 1928.

Harold Vincent Joslin, the son of William C. and Elizabeth Florine (Freeman) Joslin, was born on June 22, 1883, at Belleville, N. Y. He was educated at Clinton Liberal Institute, Fort Plain, N. Y., and at Brown University, Providence, R. I., having been graduated with the degree of Bachelor of Arts in 1904.

After graduation, Mr. Joslin's first work was with the White Mountain Lumber Company, of Conway, N. H., where he was engaged in land surveys and in the location of narrow-gauge railways. From there, he went to Newark, N. J., where he served for a short time as Draftsman with the New York and New Jersey Telephone Company. He then became associated with Betts Academy, at Stamford, Conn., as Instructor in Mathematics.

In October, 1905, he entered the service of the Norfolk-Southern Railway, in Norfolk, Va., where he won rapid promotion. He served, successively, in

* Memoir prepared by George F. Syme, M. Am. Soc. C. E.

the positions of Draftsman, Levelman, Transitman, Resident Engineer, Assistant Engineer of Maintenance of Way, and First Assistant to the Chief Engineer, until March, 1910. He then accepted a position with the City of Norfolk, and was in charge of surveys for water supply until June, 1911.

From June, 1911, until March, 1912, Mr. Joslin served as Construction Engineer for the Yadkin River Power Company, a subsidiary of the Carolina Power and Light Company, of Raleigh, N. C., on the construction of a hydro-electric plant at Pee Dee, N. C.; this work included the design and layout of the plant. He was then engaged by the Norfolk-Southern Railway Company for a few months as Locating Engineer on its Raleigh-Charlotte Branch. At this time he re-entered the service of the Yadkin River Power Company and the Carolina Power and Light Company, as Assistant to the Construction Manager and Cost Engineer and served in this capacity until April, 1913, when he went to Grace, Idaho, as Assistant to the Construction Superintendent and Cost Engineer of the Phoenix Construction Company. In this capacity he was engaged on hydro-electric surveys, design and installation of plant and subsidiary structures, and general construction.

In August, 1914, he returned to the Carolina Power and Light Company and Allied Companies, at Raleigh, where he had charge of the design and construction of substations, transmission lines, and miscellaneous work of a similar nature, until May, 1918, when he was appointed General Field Cost Engineer for the United States Housing Corporation, at Washington, D. C. He served one year in this capacity and another year as Manager of the Wilson Housing Corporation, of Wilson, N. C.

Mr. Joslin then returned to Raleigh and became associated with the North Carolina State Highway Commission as Assistant to the Chairman. He served this Commission until October, 1923, when he resigned to return to the Carolina Power and Light Company, for which he handled the design of transmission lines, bridges, buildings, and other structures, and also supervised their location and construction, as well as the surveys and investigations for water-power development. He held this position at the time of his death, on November 3, 1928.

Mr. Joslin was a faithful communicant of the Protestant Episcopal Church, a member of Phi Beta Kappa Society, Delta Kappa Epsilon Fraternity, and the William G. Hill Masonic Lodge No. 218, of Raleigh.

He was a member of the North Carolina Society of Engineers and of the Raleigh Engineers' Club, having served the latter as Vice-President. He took a very active part and a deep interest in all matters pertaining to these local engineering societies, and whenever possible attended all their meetings.

He ably served on many engineering committees appointed to co-operate with various civic bodies in the handling of local public problems. His advice was eagerly and frequently sought by his contemporaries, who appreciated and respected his sound judgment on technical matters, and the ever-present friendliness with which he responded.

Mr. Joslin was well known professionally and socially throughout his adopted State, and his memory will live in the hearts of his friends, until

they, too, answer the last call. After that, the enduring works he left behind will direct future generations to the accomplishments of an engineer, which time cannot erase. "By their works shall ye know them."

He was an outstanding figure in his community, and his lovable disposition, courtly manners, and splendid character endeared him to all who were privileged to know him. His untimely passing has spread universal sorrow among his friends and is a distinct loss to his employers and to his profession.

Mr. Joslin was married on December 1, 1908, to Annie Devereux Hinsdale, of Raleigh, who, with four children, Ellen Devereux, Devereux, William, and John Hinsdale, survives him.

Mr. Joslin was elected a Junior of the American Society of Civil Engineers on April 6, 1909; an Associate Member on February 6, 1912; and a Member on October 8, 1918.

OSCAR FRANCIS LACKEY, M. Am. Soc. C. E.*

DIED DECEMBER 19, 1928.

Oscar Francis Lackey was born in Washington, D. C., on July 1, 1874. He was the son of the late Milford Fiske Lackey, who served for many years as Chief Auditor of the Treasury Department. The family moved to Baltimore, Md., when Oscar was thirteen years old. He was educated at Rock Hill College and was graduated from Johns Hopkins University in 1896. In 1897, he went to Santiago, Chile, as an Engineer of the War Department under the late Gen. Leonard Wood, U. S. A. He survived an attack of yellow fever and returned to Cyba on dock construction work and, later, went to Panama on an engineering engagement connected with the construction of the Panama Canal.

Mr. Lackey returned to the United States in 1905 and was appointed Principal Assistant to the late N. H. Hutton, M. Am. Soc. C. E., who was then Chief Engineer and President of the Harbor Board of Baltimore. When Major Hutton died, a few years later, Mr. Lackey succeeded him in this important post and had charge of important dock and pier construction. He was one of the first, if not the first, engineer to utilize reinforced concrete piles in pier construction. He held this position until 1915, and his extensive harbor development plans and recommendations for a \$50 000 000 loan for this purpose were looked upon as fanciful at the time. Later developments, since the end of the World War, have thoroughly justified Mr. Lackey's vision, and a loan of this amount has been passed by the Maryland Legislature and voted by the people.

In 1912, Mr. Lackey was selected as one of the five Directors of the Association of Seaport Authorities and, in 1914, he was elected President of the Association of Port Authorities.

In 1915, he became Assistant to the President of the Poole Engineering Company, of Baltimore, and in this position negotiated the purchase of a

* Memoir prepared by Ezra B. Whitman, M. Am. Soc. C. E.

manufacturing plant in Lafayette, Ind., engaged in the manufacture of shells for the British Government; he operated the plant for two years. In 1918, he was appointed Supervising Engineer in the War Department, under the late Maj.-Gen. George Goethals, U. S. A., M. Am. Soc. C. E., for the construction of the Atlantic Port terminals in Boston, Mass., Philadelphia, Pa., Brooklyn, N. Y., Norfolk, Va., New Orleans, La., and Charleston, S. C.

Mr. Lackey returned to Cuba on consulting engineering work in 1921 and, in 1922, he went to Colorado Springs, Colo., as Vice-President and General Manager of the Colorado Springs Light, Heat, and Power Company. On the change of ownership of this Company, he returned to the East and was engaged as Consulting Engineer on several dock and pier projects. In 1924, in addition to his consulting work, he was appointed head of the Transportation Bureau of the City of Baltimore, which position he held until 1927. Shortly before his death, he was engaged by the State Roads Commission of Maryland to pass upon the plans of the proposed \$10 000 000 bridge across Chesapeake Bay.

While in Panama, Mr. Lackey was injured on the work, and although this injury troubled him until the time of his death, his indomitable spirit kept him going when men of lesser courage would have given up. He was of the stuff that never yields to misfortune, but turns defeat into victory. He was intensely loyal to his friends and was ready at all times to fight for what he believed to be right. His uncompromising spirit, his undeviating loyalty to high personal principles and to professional standards marked him as a man in whom others could put their trust. To his professional work, he gave not only his head, but his heart, and whatever he did was done with all his might. He was never lukewarm nor a slacker in anything, and he never failed to make good where it was humanly possible to do so.

Mr. Lackey died in Baltimore on December 19, 1928, of pneumonia, after an illness of less than a week. His death was a great shock to his many friends and came only a few weeks after his appointment by Governor Ritchie to the important position as Chairman of the State Roads Commission of Maryland.

Mr. Lackey was elected a Member of the American Society of Civil Engineers on April 6, 1909.

FRANK McDOWELL LEAVITT, M. Am. Soc. C. E.*

DIED AUGUST 6, 1928.

Frank McDowell Leavitt was born at Athens, Ohio, on March 3, 1856. He was the second son of the Rev. McDowell Leavitt, and Bithia (Brooks) Leavitt. His paternal grandfather, Humphrey Howe Leavitt, as a young lawyer, had moved from Eastern Massachusetts to Southern Ohio and later

* Memoir prepared by Logan Cresap, Commander, U. S. N. (Retired), New York, N. Y.

became, in turn, Congressman for the Cincinnati District and Judge of the United States District Court. His maternal grandfather, Moses Brooks, when a young man, had moved from Western New York to Cincinnati, Ohio, where he became a substantial merchant and then a prominent member of the bar of that city.

In 1868, Mr. Leavitt's father moved to New York, N. Y., and subsequently became President of St. John's College, at Annapolis, Md., and of Lehigh University. On arrival in New York, young Frank was entered in the famous Hunter Public School on 13th Street, but, in 1869, the lad was forced to leave because the family moved to Orange, N. J. After two years of school in Orange during which he had shown the most pronounced predilection for engineering, and decided qualities of self-discipline, industry, and self-reliance, he entered Stevens Institute of Technology from which he was graduated in 1875. His Alma Mater conferred on him the degree of Mechanical Engineer in 1905, and the degree of Doctor of Engineering, on the Fiftieth Anniversary of the Institute, in June, 1921.

Immediately after graduation, Mr. Leavitt entered the employ of the late Fred E. Sickels, M. Am. Soc. C. E., a prominent engineer of that day. One of his first assignments was the development of a steam steering gear for the U. S. S. *Trenton*, which vessel was lost in the great Samoan hurricane about 1888.

In August, 1876, he became Head Draftsman for Bliss and Williams, of Brooklyn, N. Y., and was employed there until early in 1881. He then decided to try other branches of engineering and for nearly a year was Master Mechanic of the Texas Division of the Mexican National Railroad, with headquarters at Corpus Christi, Tex. Late in 1881, he returned to New York and became Superintendent of the Graydon and Denton Manufacturing Company, of Jersey City, N. J., manufacturers of rock drills and mining machinery. He remained in this work for two years and patented a number of important improvements in the tools he was then manufacturing.

In 1883, however, Mr. Leavitt withdrew from the mining machinery field and returned to the E. W. Bliss Company (successor to Bliss and Williams), as Assistant Superintendent, and, in 1884, was appointed Superintendent of that Company. From this time until about 1900, he was actively engaged in designing and manufacturing a most varied line of sheet-metal working machinery.

The records of the Patent Office commenced to show his name as an inventor within a year or two of his graduation from Stevens Institute, and from that time until 1921 or 1922, a constant stream of patents aggregating approximately three hundred, testify to his mechanical skill and inventive genius. The outstanding accomplishment of the early years of this productive life was the invention and development of the first automatic tin can body-making machine. Until that time, such tin cans as were used in the United States were made practically by hand and to Mr. Leavitt belongs the honor and distinction of having been the first man to conceive and develop a machine which would perform all the necessary operations in the production of such can bodies in a fully automatic machine. At that time the rate of speed at

which this machine made these bodies was considered tremendous and so far beyond the normal requirements of can users that much difficulty was encountered in selling it. Meeting little or no encouragement in this country the machine was sent to the Paris Exposition about 1880 in the hope that some of the large canners of England or Continental Europe would appreciate its economic value. Unfortunately, however, only failure met this effort and the machine had to be brought back to Brooklyn and for a time was regarded practically as a "white elephant". Finally, however, one of the large can makers and packers in the Chicago District became interested and purchased it. Its use gave a tremendous impetus to canning in America and very properly may be regarded as the first stage in the modernization of that industry.

Another machine of Mr. Leavitt's which soon came into wide and ever-expanding use was the toggle drawing press for use in the manufacture of all sorts of hollow drawn ware, kitchen utensils, etc. His early inventions in drawing and stamping machines were the initial steps in the development of the huge power presses used to-day, particularly in this country, in the automobile industry for the quantity production of automobile bodies, frames, and accessories.

In addition to these two specific machines, on which improvements naturally were made later, Mr. Leavitt patented many original types of other machines, such as, drop hammers, punching presses, clutches, air compressors, shaft couplings, pumps, etc.

In the late Eighties, the Navy Department was making efforts to introduce into the United States the manufacture of up-to-date ordinance. These efforts resulted in the purchase of many European patent rights by American firms and the introduction of the manufacture of the latest types of armor-plates, armor-piercing projectiles, built-up guns, and automobile torpedoes into American industry. The Bethlehem Steel Company, the Carpenter Steel Company, and the E. W. Bliss Company were among the important commercial pioneers in this movement.

After an extensive tour abroad, during which he visited the British, Austrian, and German torpedo and projectile manufacturing plants, Mr. Leavitt, representing the E. W. Bliss Company, purchased the American rights to the Whitehead torpedo and, in 1890, introduced its manufacture into America. To the surprise of the vendors, he purchased no patent rights or machines involved in the complicated manufacture of the torpedo, but immediately on his return, set to work to develop his own manufacturing equipment. In this, he was so successful that the original manufacturers were soon outstripped in economical and accurate production. During this same period he designed and installed the plant of the United States Projectile Company, in which small and medium caliber "common" projectiles were manufactured.

Subsequent to 1890 Mr. Leavitt's activities were gradually shifted from metal-working machines to automobile torpedoes and from 1900 until just prior to the World War, the Patent Office records show a steady stream of developments in torpedoes under his name. When he first undertook the

manufacture of the Whitehead torpedo, it was a puny bronze instrument about 11 ft. long and 17 in. in diameter, with a range of about 800 yd., uncertain directive control, and carried an explosive charge of about 50 lb. of gun cotton. Mr. Leavitt gradually improved the torpedo by the introduction of steel for the air flask, the Curtis turbine for propulsion, the combustion of fuel in the air supply, the generation of steam in connection with that combustion, an entirely new and dependable gyroscopic steering apparatus, and many other innovations. His development of the gyroscope, alone, was an important contribution to mechanical engineering, and found application in other fields.

In order to secure the time to assemble and properly develop his ideas for this new torpedo, he left the E. W. Bliss Company shortly after 1900, and for a year or more established his own office and drafting-room in New York. On the issue of the patents for this new torpedo and their purchase by the E. W. Bliss Company he returned to that Company as an Officer and Director. This new torpedo became known as the Bliss-Leavitt torpedo and from about 1910 until the present time it has been, and still is, the standard equipment of the United States Navy. At the beginning of the World War this torpedo was about 21 ft. long and 21 in. in diameter, capable of reliable and accurate control over a range of about 13 500 yd. It was propelled by a super-heat steam and combustion gas turbine, operating at a temperature of about 1 000 to 1 100° Fahr., the most efficient torpedo in any navy, and carrying an explosive charge of more than 200 lb. of tri-nitro-toluol.

At the outbreak of the World War Mr. Leavitt offered his services to the Navy Department and shortly thereafter was placed in charge of a small committee known as the "Committee on Experimental Power", to which the Bureau of Steam Engineering assigned the task of developing a steam plant for the propulsion of aircraft. For this work, Mr. Leavitt was commissioned on a dollar-a-year basis. Having by this time practically retired from active connection with the E. W. Bliss Company, except as Director, he could devote his entire time to this work, which he proceeded to do with his usual vigor. At the end of the war the Navy Department decided to continue the work of this Committee, as it was believed that although the boiler plant which had been developed might not actually become a competitor of the gasoline engine, for aircraft use, such promising and interesting features had been produced or, were under investigation, which possibly might have a much wider application in naval engineering, particularly in submarine propulsion, as to warrant the continuation of the experimentation. The Committee, therefore, was reduced in size and was authorized to complete its investigation under Mr. Leavitt's direction. It continued to function for nearly two years, and when finally dissolved, turned over to the Navy Department plans and test records for an aircraft boiler which, with all appurtenances, control apparatus, and the necessary water, delivered about 1 000 h. p., with a total weight of about 2 000 lb. This was practically the last engineering work which Mr. Leavitt undertook.

He was known in the Engineering Profession, and particularly among his friends and associates in that profession, as a genius in conceiving and

designing mechanism to accomplish, with the utmost simplicity, a given purpose or necessity. His remarkable ability to visualize a problem, and his knowledge of mechanical motion and force, enabled him to picture the most difficult mechanism in the simplest of forms, to design its parts in detail, and to direct its construction in the shop, oftentimes without even assembling the parts on drawings.

Mr. Leavitt was a gentle, modest, and friendly man, simple in his tastes and personal requirements, and these qualities endeared him to those who knew him personally, and claimed, and held, the respect of his business associates. His principal diversions were painting in oils and reading. He was an omnivorous and broad reader, possessing the happy faculty of retaining what he read, and the profundity of his knowledge, when unconsciously revealed, was a source of wonder and enjoyment to his friends. In his personal relationships, his outstanding quality was a kindness and willingness to help others, with that help which relieves the necessity, and the manner that graces the act.

In 1893, Mr. Leavitt was married to Gertrude Mitchell Goodsell, of New York, the daughter of Charles Morgan and Elizabeth Mitchell Goodsell. He is survived by his widow and a daughter, Beatrice Leavitt.

He was a member of the American Society of Naval Architects and Marine Engineers, the American Society of Mechanical Engineers, and the Engineers' Club of New York, the University Club of Washington, D. C., the Riding and Driving Club, and the Montauk Club, of Brooklyn, the Scarsdale Golf Club, of Scarsdale, N. Y., and the Sankaty Head Golf Club, of Siasconset, Mass.

Mr. Leavitt was elected a Member of the American Society of Civil Engineers on December 2, 1885.

GEORGE ALBERT NOSKA, M. Am. Soc. C. E.*

DIED NOVEMBER 12, 1928.

George Albert Noska was born on April 5, 1873, in Philadelphia, Pa. He was educated in the public schools of that city and, later, attended the Philadelphia Manual Training School.

In 1891, Mr. Noska started work with the Pencoyd Bridge Company, with which he remained until 1897, when he entered the employ, as Steel Designer, of the late George B. Post, M. Am. Soc. C. E., in New York, N. Y. He held this position until 1899, with the exception of the time he spent in service during the Spanish-American War. He was Corporal of Company M, 71st New York Volunteer Infantry, and saw service in Cuba, at San Juan Hill, and in other engagements. Subsequently, he served in the New York National Guard and was commissioned First Lieutenant of the Depot Infantry, National Guard of New York, on August 2, 1916.

* Memoir prepared by Charles L. Lange, Esq., New York, N. Y.

In 1899, Mr. Noska became associated with Francis Kimball, Architect, as Superintendent of general construction, with whom he remained until 1908, when he was employed in the offices of D. Everett Waid, Architect. In this position his time was largely taken up in passing on plans on which the Metropolitan Life Insurance Company proposed loans.

In May, 1917, when the United States entered the World War, Mr. Noska again joined the Army. He was enrolled at Madison Barracks, Sacketts Harbor, N. Y., as Student in the Cadet Engineer Corps and was assigned to the 78th Division. He was discharged from service on July 2, 1919, at Camp Dix, New Jersey, as Captain of the 303d Engineers, 78th Division. He served overseas from May, 1918, to June, 1919, and saw active service at Chateau-Thierry, Meuse-Argonne, and St. Mihiel, and was awarded two gold chevrons. After his discharge he was commissioned Major in the 1st Battalion, 303d Engineers, 78th Division, Officers Reserve Corps. He was intensely patriotic as his record shows and was 44 years old at the time of his enlistment in the World War. He refused to consider any but active service at the front and undoubtedly sowed the seed there which developed into the heart attack which caused his death.

Following his discharge in 1919, Mr. Noska became a member of the firm of Chris. J. Jeppessen, Incorporated, until 1923, when he entered into partnership with Mr. Charles L. Lange, under the firm name of Lange and Noska. He was conducting this business at the time of his death on November 12, 1928. During these latter associations involving work around New York City, he was engaged on the designs of the Canadian-Pacific Building, Saks Department Store (Fifth Avenue), the building at Broadway and 40th Street, the Williamsburgh Savings Bank, the Riverside Church, and innumerable other structures.

Mr. Noska was married, on October 9, 1905, to Margaret Peasley. He is survived by his widow and two daughters, Evelyn (Noska) Douglass and Ruth Adelaide Noska.

Mr. Noska was elected a Member of the American Society of Civil Engineers on September 5, 1911.

EDWARD JONES PEARSON, M. Am. Soc. C. E.*

DIED DECEMBER 7, 1928.

Edward Jones Pearson, the second eldest of a family of six children—three sons and three daughters—of Leonard and Lucy S. (James) Pearson, was born at Rockville, Ind., on October 4, 1863. His father followed railroading as his profession and held prominent positions as a railroad officer during his lifetime. The family was of English origin, John Pearson the earliest paternal American ancestor having come from England about 1630 to 1635, and having settled at Rowley, Mass.

After elementary preparation in the public schools of Chicago, Ill., Mr. Pearson entered Cornell University at Ithaca, N. Y., from which he was gradu-

* Memoir prepared by C. E. Smith, M. Am. Soc. C. E., and M. K. Dugan, Esq., New Haven, Conn.

ated with the degree of Civil Engineer in 1883. He was a member of the Delta Upsilon Fraternity.

He began his railroad work during the summer vacation of 1881, as a Rodman in the Engineering Department of the Missouri Pacific Railway Company, on the extension from Atchison, Kans., to Omaha, Nebr., and also on construction work in Indian Territory and in Texas. In the summer of 1882 he was engaged as Assistant Engineer in terminal construction work at Portland, Ore., and after graduating in 1883 was employed by the Northern Pacific Railway Company as Supervisor of Track, until 1884, when he was placed in charge of construction train service. From 1890 to 1892 he was Division Engineer on lines east of Livingston, Mont., and was then made Principal Assistant Engineer of the Wisconsin Central Railway Company, the Chicago and Calumet Transfer Company, and the Chicago Terminal (leased by the Northern Pacific Railway Company), in charge of terminal construction.

Up to this time Mr. Pearson's work had been engineering exclusively. He had always shown, however, a keen interest in, and the desire to be associated with, operation and in 1894 was appointed Superintendent of the Yellowstone Division. He served the Northern Pacific Railway Company as Superintendent of the Rocky Mountain and Pacific Divisions until 1902, when he was made Assistant General Superintendent. In 1903, he was appointed Acting Chief Engineer, and, in 1904, Chief Engineer.

Mr. Pearson continued in this work until December, 1905, when he took charge as Chief Engineer of the Pacific Coast Extension of the Chicago, Milwaukee and St. Paul Railway, a line through rugged country, 1350 miles long, which he built in as many days. He organized and started the operation of this extension. In 1911, he was appointed First Vice-President of the Missouri Pacific and the St. Louis, Iron Mountain and Southern Railway Companies, in charge of operation, thus returning to the first railroad on which he had served as a Rodman thirty years before.

Mr. Pearson was associated with the Missouri Pacific Lines until April, 1915, when he left to become First Vice-President in charge of the Texas and Pacific Railway. During his four years of service with the Missouri Pacific System, the physical condition and capacity of the property was greatly improved, its operation reorganized, and its earning power brought to a much higher level.

He remained with the Texas and Pacific Railway Company only about a year, that is, until March, 1916, when he was called to the service of the New York, New Haven, and Hartford Railroad Company, as Assistant to the President, and Vice-President. He became President of the Company on May 1, 1917, on the resignation of the late Howard Elliott, Affiliate, Am. Soc. C. E. During the period of Federal control, Mr. Pearson was Federal Manager of the Railroad, following which he was again elected President.

When Mr. Pearson entered its service, the New Haven Railroad was ill-equipped for handling the large volume of freight business being offered. He remodeled the road from a combination of many inadequate separate yards and engine terminals, to a consolidated system with adequate terminals at

central points. Heavy locomotives were provided, bridges strengthened, passing tracks lengthened to enable the handling of modern trains, and its operations were completely reorganized. Dividends, which had been suspended in 1913, were resumed shortly before his death.

It is worthy of note that, largely because of his caution in railroad operation, not one passenger was killed in a train accident on the New Haven Lines during twelve and one-half years of his administration. Mr. Pearson's service to the New Haven Railroad Company and to the territory it served is fitly expressed in the following quotation from the remarks of Dr. A. T. Hadley, Director of the New Haven Railroad, and President Emeritus of Yale University, at the funeral service:

"Never was a man faced with a more discouraging task than that which confronted this man in 1920. Only those who were at his side in those dark days can know the burdens which he was called upon to carry, and the superb way in which he bore them. Step by step, with infinite patience and undaunted courage, he brought order out of chaos in the operations and finances of the New Haven System and put it in a position to serve the community under present-day traffic conditions."

The outstanding traits of Mr. Pearson's character were his desire to avoid publicity, his modesty as to his accomplishments, and his kindly consideration and loyalty for his associates, particularly those junior in authority—his desire that they should be given every opportunity for advancement and full credit for good work well done.

In June, 1926, the degree of Master of Arts was conferred upon Mr. Pearson by Trinity College. He was a member of the American Railway Engineering Association, the Cornell Society of Civil Engineers, and the Connecticut Society of Civil Engineers; he was also a Director of the American Railway Association. He belonged to the Bankers, the Engineers, and the Metropolitan Clubs of New York, N. Y.; the Graduates and Quinpiack Clubs, of New Haven, Conn.; the Metropolitan Club of Washington, D. C.; the New England and New York Railroad Clubs, and the Traffic Club of New England.

Mr. Pearson was married to Gertrude S. Simmons at Evanston, Ill., on June 7, 1899. His widow and a son, Harlow S. Pearson, survive him; also his mother, two sisters, and a brother, Raymond A. Pearson, President of the University of Maryland.

Mr. Pearson was elected a Member of the American Society of Civil Engineers on December 4, 1907.

WILLIAM GUNN PRICE, M. Am. Soc. C. E.*

DIED JULY 6, 1928.

William Gunn Price was born in Knoxville, Pa., on July 6, 1853. His father, William Price, was a physician of Welsh and English descent. His mother, Tryphene R. (Gunn) Price, was of Irish and Scotch ancestry. He

* Memoir prepared from information on file at the Headquarters of the Society.

gained his early education in the public schools of New York State, at Hartwick Seminary, and later, at Columbia University.

Mr. Price's engineering experiences began when he was eighteen years of age, in the vicinity of New York and, from 1879 to 1896, he was United States Assistant Engineer, in charge, successively, of surveys on the Mississippi River, the improvement of the Harbor of New Orleans, La., the rectification of the Red, Mississippi, and Atchafalaya Rivers, and other important works. He had charge of the construction of the two dams at the head of the Atchafalaya River, which prevented the Mississippi from flowing to the Gulf by that shorter route, in the disastrous flood of 1927. He also assisted in the first survey of the Mississippi, which previously had not been mapped accurately, and he took part in a similar survey of the Missouri River.

Mr. Price was the inventor of the electric current meter and the acoustic current meter for measuring the flow of water. Both these instruments are extensively used by engineers all over the world. At New Orleans, he designed a new and successful system of spur-dikes for protecting the banks of the river. He was also the originator and advocate of the plan for the improvement of the Lower Mississippi River, by the method which utilizes the forces of the river to dig the foundation for permanent channel-controlling works, which plan was successful as applied by him in New Orleans Harbor, the Atchafalaya River, and elsewhere. This method of construction is such that the force of the river can only sink the structure deeper in the sand while more material is being added on the top. All his inventions and improvements in engineering construction have been boldly original.

In 1896, Mr. Price became Chief Engineer for the Chicago City Railway Company, and produced the first railway track test car. This car, when run over the road at train speed, using five pens on a moving sheet of paper, recorded the location and amount of all defects in the track.

The same year he originated the momentum power brake for street cars, which was used on all the Chicago lines and by many other railways, until the advent of much larger cars necessitated the use of the air-brake. Mr. Price also invented an automatic fare box for street cars which prevented drivers from "knocking down" fares.

For a period of ten years, beginning with 1903, he was employed by the Standard Steel Car Company, of Pittsburgh, Pa., to design improvements in railway equipment. The Standard Motor Truck Company, a subsidiary Company, was incorporated especially to control the manufacture and sale of Mr. Price's inventions. During these ten years he placed with the Standard Motor Truck Company, on royalty, thirty-three patents of railway equipment, many of which are in extensive use in the United States and other countries.

In 1913 Mr. Price went to Yakima, Wash., and contributed to labor-saving devices in the fruit industry, such as, a fruit-sizing machine; an automatic elevator, which, in the warehouses, saves the work of several men; box-making machines; box presses; an improved gravity conveyor; a machine for sizing potatoes; and a box press for dried apples. Not the least interesting of his inventions, is an ingenious little vest-pocket pencil sharpener. His last invention was a hydraulic friction brake for automobiles.

Mr. Price had not been in the best of health for two and a half years prior to his death, having a leakage of the heart; but in the late spring and early summer of 1928 he was feeling so much improved that on June 20 he started with his wife on a trip to Central New York. They stopped en route at the Battle Creek Sanitarium for a physical examination. Receiving no benefit from a two weeks' stay at the institution, Mr. and Mrs. Price resumed their journey. Mr. Price's condition became serious, however, and at Detroit, Mich., he was taken from the train to the Fort Shelby Hotel, where he died the next day, Friday, July 6, 1928, the seventy-fifth anniversary of his birth.

He is survived by his wife, Mary Kelley Price, a native of Schenewus, N. Y., to whom he was married in Chicago, in 1879; one son, William Kelley Price, of Selah, Wash., and one grandson, William Griffin Price.

Mr. Price was a member of the American Association of Engineers, Professional Engineers, and the American Association for the Advancement of Science; he was also a Mason and belonged to the Scottish Rite and the Shrine.

Mr. Price was elected a Member of the American Society of Civil Engineers on April 3, 1895.

ARTHUR BREESE PROAL, Jr., M. Am. Soc. C. E.*

DIED AUGUST 22, 1922.

Arthur Breese Proal, Jr., was born on October 10, 1869, in New York, N. Y., the son of Arthur Breese Proal, a prominent banker and financier of New York. His mother was Sydney Mathiot, of Zanesville, Ohio, the daughter of the Hon. Josiah Mathiot.

Mr. Proal received his education at Stevens Preparatory School and was graduated from Pratt Institute in Brooklyn, N. Y. He began his first practical work in 1892 in the Electrical Testing Department of the Metropolitan Telephone Company, which later became the New York Telephone Company. He was engaged mostly in laying underground cable systems in New York City and assisted in the laying of the first cable under the Hudson River.

In December, 1892, he was engaged as Construction and Mechanical Engineer for the Newark Electric Light and Power Company of Newark, N. J. (now the Public Service Corporation of New Jersey). In this capacity, he had charge of the design and erection of steam and electric equipment, as well as the construction of several large power houses, together with detailed work for docks and intakes on the water-front.

In November, 1896, Mr. Proal was made Superintendent and Chief Engineer of the New Haven Street Railway Company, at New Haven, Conn., for which he had charge of the entire operation of the System and the extension of its service. During this period, he laid out and built a number of miles of interurban trolley line and acted as one of the pioneers in the construction and operation of Light House Park at the end of the Interurban Line.

* Memoir prepared from information on file at the Headquarters of the Society.

In January, 1898, he became Superintendent and Chief Engineer of the New York and Staten Island Electric Company, which, in addition, controlled the Southfield Beach Railway Company. Mr. Proal built the latter road complete, including its entire power equipment. He was also Superintendent and Chief Engineer of the New York and New Jersey Ferry Company, the Richmond Light, Heat, and Power Company, and of several smaller companies. At this time he designed and supervised the construction and erection of the central power plant—7 000 h.p.—that furnishes all the lights and railway current in Richmond County, New York.

In January, 1901, Mr. Proal became associated with the Robins Conveying Belt Company, of New York, as Sales Engineer, with which company he remained until his death from pneumonia on August 21, 1922. He was ill but a few days.

Mr. Proal was the inventor of many devices used in the art and construction of conveyances and was particularly prominent in the planning and design of many of the important vessel-loading plants on the Atlantic Coast. He also developed and patented two types of bridge tramways for open storage of materials, which are in successful operation throughout the world to-day. These plants stand as worthy memorials to his remarkable character and ability.

Coupled with his exceptionally observant and perceptive nature, Mr. Proal had a very retentive memory and supplemented his extended engineering knowledge with the rare ability of being able to transmit this knowledge to others concisely and freely. His death was mourned by his associates not only because of his cheerful disposition, his great mechanical ingenuity, and his sales ability, but also because of his exceptional fund of knowledge and his willingness to impart it. He left a vacancy impossible to fill.

In October, 1906, Mr. Proal was married to Sara Francisco Harris, of Newark, N. J., who survives him.

Mr. Proal was elected a Member of the American Society of Civil Engineers on March 7, 1906.

EDWARD EMMET SANDS, M. Am. Soc. C. E.*

DIED OCTOBER 27, 1923.

Edward Emmet Sands was born in Columbus, Ohio, on January 5, 1877, the son of Louis K. and Emily Sands. His early education was received in the grade and High Schools of Sparta, Wis., and his technical education at the University of Wisconsin, from which he was graduated with a degree of Bachelor of Science in Civil Engineering in 1900. This was followed by a Civil Engineer degree in 1906.

During the winters of 1900 and 1902, Mr. Sands served as Instructor in Civil Engineering at his Alma Mater; during the summers he acted as City Engineer of Sparta, designing and supervising water, sewer, and paving improvements. From 1902 to 1908 he acted as Engineer in the United States

* Memoir prepared by John B. Hawley, M. Am. Soc. C. E.

Reclamation Service and from 1908 to 1911, as Manager for the Upper Columbia Irrigation Company (part of 1911 and 1912 with the United States Reclamation Service). From April, 1912, to September, 1913, he served as Supervising Engineer for the Canadian Pacific Railway Company on irrigation and municipal projects in Calgary, Alberta, Canada.

In September, 1913, Mr. Sands became City Engineer of Houston, Tex., engaged in general city work and port development, the latter estimated at \$3 500 000. During this service he designed and supervised the construction of the first "activated sludge" sewage disposal plant in America. During the World War he also acted as Supervising Engineer for Camp Logan and Consulting Engineer for the Ellington, Carstner, Brooks, and Kelly Aviation Fields.

From March, 1918, to March, 1920, he was engaged as Consulting Engineer for the Atlantic, Gulf, and Pacific Company, in New York, N. Y.; from March to September, 1920, he served as President of the Houston Construction Company, at Houston; and from September, 1920, to June, 1921, as Chief Engineer for the H. F. Friestedt Company, Chicago, Ill., and Houston.

In June, 1921, Mr. Sands entered private consultation practice in partnership with John B. Hawley, M. Am. Soc. C. E., of Fort Worth, Tex., with offices in Houston and Fort Worth. This firm designed and supervised the construction of water purification and sewage disposal works at Fort Worth, Breckenridge, Lubbock, and Sweetwater, Tex., and port improvements at Corpus Christi, Tex., totaling several million dollars, during 1921, 1922, and 1923.

In 1903, Mr. Sands was married to Isabel Garrison, of Denver, Colo. They had four children, Grace Isabel, Ruth Elizabeth, Emily Ann, and Edward Emmet, Jr.

At the zenith of his mental and professional activities, early in 1923, he was stricken with cerebral tumor; two operations failed to abate the malady, and while visiting his father at Milwaukee, Wis., the end came as he slept, on October 27, 1923.

Edward Sands was known to his profession as an unusually erudite, skillful, practical, experienced, efficient, trustworthy engineer; to his engineering associates, as a sound scientist, able executive, and wise counsellor; to his social acquaintances and friends, as a genial, companionable, helpful comrade; to his family, as a considerate, loving husband and indulgent, doting father; to the writer, who will ever mourn his passing, as a dear friend and faithful coadjutor.

He was a Past-President of the Texas Section of the Society; Chairman of the Houston City Plan Commission, and a Member of the following organizations: The University Club of Houston and the Houston Country Club; the Wisconsin Division of Sons of the American Revolution; Trinity Protestant Episcopal Church, of Houston; the Masonic Fraternity, having been a member of Hiram Lodge No. 5 of Madison, Wis.; and the Shrine, Arabia Temple, Houston.

Mr. Sands was elected a Member of the American Society of Civil Engineers on February 4, 1913.

JOHN TRUESDALE STEWART, M. Am. Soc. C. E.*

DIED JUNE 9, 1928.

John Truesdale Stewart was born at Loda, Ill., on January 13, 1868, the son of William R. and Nancy (Barr) Stewart. He was of Scotch-Irish descent, his ancestors having emigrated to the United States before the Revolutionary War. One great-grandfather served in the War of the Revolution and another in the War of 1812.

Mr. Stewart was graduated from Grand Prairie Seminary, Onarga, Ill., in 1888. In 1893, he was awarded the degree of Bachelor of Science from the University of Illinois, Urbana, Ill., and, in 1909, that of Civil Engineer from the same University.

While Mr. Stewart was known principally as an authority on agricultural drainage, his activities took him into other engineering fields in various parts of the country. Beginning his engineering career as Rodman on land drainage work in 1890, he spent the years 1891 to 1892 on railroad work. From 1893 to 1897 he was engaged in general engineering practice in Illinois. During the following two years he was occupied in both drainage and railroad work, holding several positions during this period. In 1899, Mr. Stewart was appointed Field Assistant with the United States Geological Survey, a position which he held until 1904, when he became Drainage Engineer, in the United States Department of Agriculture, on drainage and irrigation work in South Dakota, North Dakota, Minnesota, Arkansas, and Florida.

In 1908 he was made Professor and Chief of the Division of Agricultural Engineering, at the University of Minnesota, Minneapolis, Minn. At this time he conducted research work on the drainage and development of peat lands, the durability of drain tile, and the efficiency and maintenance of open ditches; he also prepared as sole or co-author numerous papers, several of which have been published by the United States Department of Agriculture. Among these are: "Drainage of the Eastern Parts of Cass, Traill, Grand Forks, Walsh, and Pembina Counties, North Dakota";† "Farm Drainage, Minnesota Extension";‡ "Installation of an Experimental Drainage System, Minnesota Experiment Station";§ "Review of Drainage Legislation";|| and numerous other papers on land drainage. In addition, he was senior author of "Engineering on the Farm".

Imbued with a strong sense of loyalty and patriotism, Professor Stewart was active in the military service of the United States. In 1893, he was made Brevet Captain, the Illinois National Guard. With the outbreak of the Spanish-American War, he organized a volunteer company of which he was elected Captain. He became a Major in the Engineers Reserve Corps in

* Memoir prepared by the following Committee of the Northwestern Section: J. A. Childs, Chairman, Adolph F. Meyer, and E. V. Willard, Members, Am. Soc. C. E.

† *Bulletin 189*, Office of Experiment Stations, U. S. Dept. of Agriculture.

‡ *Bulletin 13*, Office of Experiment Stations, U. S. Dept. of Agriculture.

§ *Bulletin 110*, Office of Experiment Stations, U. S. Dept. of Agriculture.

|| New York State Library, 1905-08.

1917, and served as Lieutenant-Colonel of Engineers in 1918. Although much against his desire he was not permitted to serve overseas, he distinguished himself so as to receive honorable mention from his superior officers. He held a commission as Colonel in the United States Engineers Reserve Corps at the time of his death.

Following the close of the World War, Colonel Stewart was connected with the Portland Cement Association. In 1922, he resumed his consulting practice, as a recognized authority on drainage and wet land development.

Always courteous, friendly, and interested, Colonel Stewart had many friends, both in and out of the engineering fraternity. He was a student of natural history, always fond of outdoor life, and he especially enjoyed hunting and fishing.

He was a member of the Society of American Military Engineers, the Western Society of Engineers, the American Society of Agricultural Engineers, the Engineers' Society of St. Paul, the American Legion, the Sons of the Revolution, the Society of the War of 1812; and Tau Beta Pi, Sigma Xi, Alpha Zeta, and Gamma Sigma Delta Fraternities.

On January 1, 1900, he was married to Ida Belle Wilson who survives him. He died June 9, 1928, and was buried in Arlington National Cemetery, with full military honors.

Colonel Stewart was elected an Associate Member of the American Society of Civil Engineers on September 6, 1905, and a Member on October 10, 1916.

FRANCIS REPETTI WELLER, M. Am. Soc. C. E.*

DIED JANUARY 13, 1929.

Francis Repetti Weller was born in Washington, D. C., on February 16, 1880, the son of Michael I. and Rita (Repetti) Weller. He was educated at Mount St. Mary's College, near Emmitsburg, Md., and then attended the Columbian University, at Washington, D. C., which is now known as George Washington University. He was graduated from Columbian University with the degree of Bachelor of Science in Civil Engineering in 1899, and in June, 1900, received the degree of Civil Engineer.

From 1899 to 1903 Mr. Weller was employed in the Engineering Department of the District of Columbia, principally on bridge design, and, in addition, was an Instructor at Columbian University. In September, 1903, he entered private practice as a Consulting Engineer, with headquarters at Washington.

Mr. Weller specialized in the design of steam-electric and hydro-electric power plants, transmission lines, and water-works plants. He also always had been closely connected with public utility work. For the Municipal Service Corporation, he designed and enlarged plants in Salem, Ohio, Alexandria and Staunton, Va., and Coatesville, Pa. He was the Consulting Engineer on the design and construction of hydro-electric plants for the City of Richmond, and

* Memoir prepared by M. X. Willberding, Pres., Francis R. Weller, Inc., Washington, D. C.

the City of Bedford, Va., the Watauga Power Company, at Bristol, Tenn., and the Virginia Alberene Corporation; he also made enlargements of the electric plants of the Keystone Power Corporation, at Ridgway, Pa., and the Potomac Electric Power Company, at Washington.

Mr. Weller was one of the pioneers in power development in Southwestern West Virginia and built the first power plant in that State at Logan. This plant was primarily owned by the General Utilities Operating Company, of Baltimore, and, later, by the Kentucky and West Virginia Power Company, and is now the property of the American Gas and Electric Company. Mr. Weller maintained his connections during all these changes of ownership and was responsible for many additions to the property, as well as transmission lines connecting it to plants at Sprigg, W. Va., and Hazard, Ky.

In 1917, and during the World War, he was employed by the Navy Department, Bureau of Yards and Docks, to design additions to a great many of the Navy Yards on both the Atlantic and Pacific Coasts. During this period he had a large designing force in his employ and made plans for extensions and enlargements at the Brooklyn, N. Y., Navy Yard, the Navy Yards at Philadelphia, Pa., and Norfolk, Va., the Submarine Base at New London, Conn., and a Naval Station at New Orleans, La., together with about ten other smaller projects. He was also Engineer for the United States Housing Corporation during this same period. In recognition of his excellent work for the Bureau of Yards and Docks he was made Lieutenant Commander in the United States Naval Reserve.

In 1923 Mr. Weller was employed as Consulting Engineer for the Virginia Western Power Company and built steel tower transmission lines from Charlottesville to Staunton, Va., revamped the power house at Charlottesville, and constructed sub-stations at both Charlottesville and Staunton. The following year he built steel tower transmission lines for the same Company from Clifton Forge, Va., to Ronceverte, W. Va., and wood pole distribution lines between Goshen, Buena Vista, Lexington, and Clifton Forge, Va.

In 1925, he organized a public utility holding company known as the Interstate Utilities Corporation. This holding company had properties in Central and South Georgia and Mr. Weller was its President, as well as the President of all its subsidiary companies. These properties were afterward purchased by the Southeastern Power and Light Company, and Mr. Weller then formed the Allied Utilities Corporation, another public utility holding company, of which he was President. The principal holdings of this Company were in the State of West Virginia. The Company was dissolved in 1927 and its properties were sold to the Allied Utilities Company, of which Mr. Weller was President at the time of his death.

In the spring of 1926, Mr. Weller was employed as Consulting Engineer by the Fitkin Interests to build transmission lines in Florida and South Georgia. These consisted of steel power and wood-pole lines from Tarpon Springs, near St. Petersburg, Fla., through Inglis and Jasper to Valdosta, Ga. The installation included a great many sub-stations, and the construction work continued for a period of approximately two years.

In June, 1928, he incorporated his engineering practice under the name of Francis R. Weller, Incorporated, and, at the time of his death, he was President of this organization.

Mr. Weller was a member of the American Institute of Electrical Engineers and a Charter Member of the Washington Society of Engineers. He took a great interest in civic and charitable organizations. He was a Director of the Washington Board of Trade and President of the Roman Catholic Charities of Washington. Mr. Weller was greatly interested in the Community Chest work and just before his death was made a Vice-President of the Washington Community Chest. His active work in the Washington Board of Trade and his engineering training gave him the Chairmanship of the Water Supply Committee of that body and through it he was instrumental in bringing about the present improved conditions of the Capital City's water supply.

Mr. Weller was a member of a great many social organizations and clubs in Washington, among which were the Knights of Columbus, the University Club, the City Club, the Rotary Club, the Racquet Club, the Congressional Country Club, and the Columbia Country Club. At the time of his death he was a Director of the East Washington Savings Bank and the Federal American National Bank. He was also a director of Edmonds Art Stone Company and the Washington Wimsatt Company.

In 1911, he was married to Sallie A. Harlow, of Alexandria, Va., by whom he is survived. He also left four children, Francis Harlow, Sallie Rita, Richard Hartman, and George Harlow Weller. Mr. Weller was always a devoted family man, and he took a great interest in his summer home which was located at Buena Vista, Pa.

He occupied the position of an outstanding citizen in the City of Washington and was greatly interested in questions pertaining to it and especially to its charity work. His passing was mourned by thousands of friends and especially by the poor and needy whom he had helped so often.

Mr. Weller was elected a Junior of the American Society of Civil Engineers on February 5, 1901; an Associate Member on November 1, 1905; and a Member on April 30, 1912.

NISBET WINGFIELD, M. Am. Soc. C. E.*

DIED FEBRUARY 24, 1928.

Nisbet Wingfield was born at "Cloverdale", the summer home of his family, in Dade County, Georgia, on September 23, 1861, the second son of Alfred and Julia (Lea) Wingfield. He attended the University of Mississippi and also the University of Tennessee from which he was graduated in 1880.

Immediately upon his graduation he engaged in railroad engineering, continuing in that field on various railroads in the South until January, 1885,

* Memoir prepared by James Houstoun Johnston, James J. Gaillard, and James E. Parker, Members Am. Soc. C. E.

when he went to Chattanooga, Tenn., as Chief Engineer of the water-works. He remained in Chattanooga until 1895, when he removed to Atlanta, Ga., where he made his headquarters while engaged on the design and construction of water-works and sewerage projects at various places.

In 1898, he was called to Augusta, Ga., to take charge of the enlargement of the City Water-Works, and, later, became City Engineer and Commissioner of Public Works, holding this position until 1919, when he engaged in private practice as Consulting Engineer. While City Engineer of Augusta, Mr. Wingfield planned and constructed a system of levees for the protection of the city against the flood waters of the Savannah River. This was the outstanding achievement of his career, and will serve as a monument to him. The *Augusta Chronicle*, speaking editorially, stated:

"The security of the city against flood waters has been proven time and time again and Nisbet Wingfield was the man who suggested a levee, who planned it, supervised its construction, and its success is too well known to need accentuating".

He also planned levees for Chattanooga and for West Point, Ga.

Among the projects constructed by Mr. Wingfield are several bridges in various parts of the South, including the Archibald Butt Memorial Bridge, at Augusta, a structure of pleasing architectural design, as well as the water-works at Augusta, Montgomery, Ala., Meridian, Miss., Greenwood, S. C., and a number of smaller cities. As a Consulting Engineer he was widely known throughout the Southeast and his opinion was sought on many large projects. During the World War he was retained by the Government as Project Engineer on the construction of Camp Hancock at Augusta, and on the housing project which was commenced for the enlargement of the Navy Yard at Charleston, S. C. At the time of his death, he was Consulting Engineer for the Savannah River Electric Company, in connection with this Company's proposed hydro-electric development of the Savannah River above Augusta.

Mr. Wingfield was a man of charming personality, kind and courteous to all men; cultured and polished, a delightful companion to old and young; and a man of unimpeachable integrity. His friend, Thomas Barret, former Mayor of Augusta, said of him that he was "a golden-hearted gentleman, an honest public official, and one of America's really great engineers; an entire city deplores his passing".

Mr. Wingfield loved his profession and was ever an honor to it. Broad-minded and liberal, he was always strict yet perfectly fair in all his dealings with those with whom he worked. The Society, in which he was always active and, in particular, the Georgia Section, in which he also took a lively interest and had served as Vice-President, have lost a valued member, and those who had the pleasure of his acquaintance and his professional association keenly regret his passing.

Mr. Wingfield was elected a Member of the American Society of Civil Engineers on May 1, 1895.

ANDREW LEWIS ACKHART, Assoc. M. Am. Soc. C. E.***DIED MARCH 11, 1928.**

Andrew Lewis Ackhart, the son of Nathan and Eva (Deyo) Ackhart, was born at Clintondale, N. Y., on April 3, 1887. He received his preliminary education in the schools of Highland and Poughkeepsie, N. Y., and, later, attended Cornell University at Ithaca, N. Y., from which he was graduated on June 22, 1911, with the degree of Civil Engineer. He supported himself at the University as a pioneer salesman of aluminum cooking utensils and by other activities.

Mr. Ackhart's first engineering engagement was as Inspector with the Farris Engineering Company, of Pittsburgh, Pa., on a concrete substructure for a 970-ft. highway bridge, which occupied his time from November, 1911, to January, 1912. Subsequently, from January to October, 1912, he held the position of Rodman and Instrumentman on extensive stadia topographical surveys with the Costa Rica and Panama Boundary Arbitration Commission.

In February, 1913, Mr. Ackhart accepted a position with Mackenzie, Mann, and Company of Toronto, Ont., Canada, in which he had charge of the design of concrete bridge foundations, retaining walls, and culverts for the Canadian Northern Ontario Railway. In May, 1913, he entered the employ of James McAlear, of Toronto, as Draftsman on heating, lighting, and ventilating installation for office buildings, hospitals, schools, and factories. From July to August, 1913, he was engaged as a Draftsman by J. H. Trommanhauser, of Toronto, on the design of machinery for a grain elevator for the Quebec Harbor Commission.

In September, 1913, Mr. Ackhart returned to Mackenzie, Mann and Company, where he served until January, 1915, as Assistant Superintendent on the construction of round houses, machine shops, pumping stations, and all other division buildings at Fittsback, Ont., Canada. His work also included the construction of tanks, and a section house, along 100 miles of the Canadian Northern Ontario Railway.

From April to June, 1915, he had charge of the construction of a private water supply and sewerage system at Clintondale, N. Y. Having been refused for service in the Army, because of a defective eye, he "did his bit" as a Head Inspector of 4½-in. shells at various munition plants throughout Ontario, for the Canadian Inspection and Testing Laboratory, Limited. From June to December, 1916, he was engaged by the Buffalo Construction and Equipment Company, of Buffalo, N. Y., in charge of 4 miles of a catenary tower concrete foundation, and bridge abutments and culverts for the International Railway Company. During January and February, 1917, he served as Draftsman with the Castner Electrolytic Alkali Company, of Niagara Falls, on structural steel design and machinery layouts; and from February to October, 1917, as Draftsman with the Hooker Electro-Chemical Company, of Niagara Falls, on the design of a pump-house and intake for a pumping station with

* Memoir prepared by Ernst G. Kaufmann, Assoc. M. Am. Soc. C. E.

a daily capacity of 24 000 000 gal., as well as on plant extension design and a complete design for the extension of hydrogen installation.

On October 8, 1917, Mr. Ackhart accepted the position of Assistant Engineer with the Eastern Concrete Steel Company, of Buffalo, where he had responsible charge of estimates. It was during this time that he supervised the erection of the New York Central Railroad Express Station and other structures, which consisted of a grade-crossing viaduct, a concrete building, etc.

In 1919, he was engaged by The Boldt Construction Company of Cleveland, Ohio, as Engineer and Estimator, and he held this position until his death on March 11, 1928.

On April 29, 1922, Mr. Ackhart was married to E. Louise Haner, who with two daughters, Sally Louise and Barbara Lucille, survives him.

Mr. Ackhart was a Knight Templar, a Shriner, and a Thirty-second Degree Mason, holding a great respect for this Order, of which he was a staunch supporter.

A man greatly admired and honored, he had a good word for all with whom he came in contact. He was a diligent worker and possessed a strong, magnetic personality.

Mr. Ackhart was elected a Junior of the American Society of Civil Engineers on March 4, 1913 and an Associate Member on September 10, 1918.

MERLIN CROSS CRAWFORD, Assoc. M. Am. Soc. C. E.*

DIED FEBRUARY 11, 1928.

Merlin Cross Crawford was born at Chilton, Tex., on August 21, 1889, the son of William B. and Alice Crawford.

He was graduated from the University of Texas at Austin, Tex., in 1912, with the degree of Bachelor of Arts, and in the following year received the degree of Electrical Engineer.

Mr. Crawford's first work after leaving the University was in the Testing Departments of the General Electric Company at Schenectady, N. Y. and Pittsfield, Mass. After having served a year with this Company he returned to his native State, where the work of reclaiming lowlands by drainage ditches and levees constructed by dragline excavators was in its infancy and where the construction of highways was creating a demand for gravel. Although he had been educated as an Electrical Engineer, Mr. Crawford foresaw the enormous amount of reclamation work in prospect and decided to cast his lot into that engineering field. He therefore joined the forces of Callahan and Crawford in the operation of gravel properties in South Texas.

His next position was with the W. E. Callahan Construction Company at Ennis, Tex., in the capacity of Superintendent in charge of a large number of reclamation projects, including parts of two of the largest projects in the State, namely, the Storage Dam near Seymour, built for the Wichita Falls Water Improvement District No. 1, and the Garza Dam, built for the City of

* Memoir prepared by E. S. Heyser, Assoc. M. Am. Soc. C. E.

Dallas. This connection continued until 1927, when he entered business for himself. At the time of his accidental death on February 11, 1928, which resulted from a fall from a hotel window in San Antonio, Tex., he was engaged in the beginning of his largest individual contract—the erection of the steel in the thirty-two story Smith-Young Tower.

Had it not been for his untimely death, Mr. Crawford would undoubtedly have made for himself, in an even greater degree, a record for achievement, as he possessed to a remarkable extent the qualities that make for success in the construction and engineering world. His energy was unbounded and he had the happy faculty of being able to build up an organization that was ever ready, through a spirit of unfaltering loyalty, to give to him the same whole-hearted support that he had always given to his employers. His cheerful, genial personality, and his unswerving devotion to his friends made him an always welcome addition to any group.

He was a member of Hella Temple, of Dallas, and the Dallas Athletic Club. He was also a member of Sigma Alpha Epsilon Fraternity.

Mr. Crawford was married in June, 1917, to Jean Figh, of Dallas, who survived him only one month. He left, in addition to his widow, two daughters, Jean and Patricia.

Mr. Crawford was elected an Associate Member of the American Society of Civil Engineers on May 8, 1922.

GEORGE CHRISTIAN SCHOENBERGER, Assoc. M. Am. Soc. C. E.*

DIED SEPTEMBER 24, 1928.

George Christian Schoenberger was born in Buras, Plaquemines Parish, La., on January 17, 1875, the son of George M. and Mary (Walker) Schoenberger. He received his early education at Buras, and subsequently entered the Louisiana State University at Baton Rouge, La., from which he was graduated in 1898, receiving a Bachelor of Science degree in Civil and Mechanical Engineering. During his college career, he was a member of the famous football team of 1896, took an active interest in the military life of the college, serving as Major during his Senior year, and was a member of the Sigma Alpha Epsilon Fraternity.

Shortly after his graduation, Mr. Schoenberger entered the employ of the State Board of Engineers of Louisiana, which, in later years, was to have him as its Chief Engineer during a most trying period.

He began his professional career as Rodman, and later, was promoted to be Levee Inspector on the Red River. After a year with the State Board, he resigned to accept a position with the Houston Irrigation Company as Assistant Engineer in charge of irrigation work in Southwestern Louisiana. This engagement was followed by short services as Rodman and Inspector with the United States Engineer Office, and as Field Superintendent with the Mississippi Construction Company, engaged in levee work.

* Memoir prepared by Gervais Lombard, Esq., New Orleans, La., and A. F. Jacobi, Assoc. M. Am. Soc. C. E.

In November, 1901, Mr. Schoenberger returned to the State Board of Engineers as Surveyor in charge of party on location and construction of levees. Just a year later, in November, 1902, he was appointed Junior Engineer with the U. S. Engineer Office, New Orleans, La., in charge of levee construction in the Lafourche and Pontchartrain Levee Districts. The following six years were spent in this capacity. During a part of this time, Mr. Schoenberger and his family made their home in Natchez, Miss., where they enjoyed the society of a host of friends.

Although Mr. Schoenberger rather frequently changed positions during this early period, his work always was related to river control, in some of its varied phases. He must have realized early that this was to be his life work, and that a broad training embodying all its branches was highly desirable. Consequently, in October, 1908, he again changed positions, this time going to the Mississippi River Commission, 4th District, District Engineer's Office, at New Orleans, with which he stayed until 1917, when he was appointed a Member of the Board of State Engineers.

During the high-water periods of 1912, 1913, 1915, and 1922, much of the responsibility for keeping intact the levee lines in the 5th Louisiana and Tensas Basin Levee Districts was Mr. Schoenberger's, and due largely to his management in rescue work after the break at Fagette, La., no lives were lost. It was during this flood that Mr. Schoenberger first began to attract attention by his consummate skill and untiring efforts.

From the time in 1917 that he was appointed a Member of the State Board of Engineers by Governor Ruffin G. Pleasant, until his death, Mr. Schoenberger was constantly with the State Board, having been re-appointed in 1920 by Governor John M. Parker, and in January, 1925, being named Chief Engineer by Governor Henry L. Fuqua to fill the vacancy caused by the death of the late Frank M. Kerr, M. Am. Soc. C. E.

The record flood of the Mississippi River and its tributaries in 1927 brought forth the sterling qualities of Mr. Schoenberger, upon whose shoulders fell a large share of the responsibility for the protection of the levees and property of thousands of people in the Lower Valley. He took a leading part in the cutting of the Caernarvon Levee below New Orleans, creating the artificial crevasse which was to relieve the danger of a break at the city above. Supervising the holding of the levees in the upper section of the State occupied every available hour of his time, and he fought to the last ditch for every foot of levee. It was only after everything that was humanly possible had been done that the several disastrous crevasses occurred. It was a losing fight at the time, but with defeat came victory, for the great disaster opened the eyes of the entire country to the need of adequate flood control.

With complete victory in sight, his quarter-century battle against the flood menace of the Mississippi River came to an end on Monday, September 24, 1928, when he died suddenly from a heart attack at his home. Although complaining of pains in the region of the heart, he had arisen as usual, dressed, and prepared to go to his office. While in the garden of his home, he was stricken and died before the arrival of a physician. Mr. Schoenberger was described as a martyr to flood-control work by his physician who pronounced the weak

condition of his heart as due to this strenuous work during the fight to save the levees in 1927, and his labors in Washington, D. C., in behalf of Federal flood control.

Public tribute was paid to Mr. Schoenberger after his death by his numerous friends and admirers, many of whom held high office. The following editorial published in New Orleans, just after his death, expresses the esteem in which he was publicly held:

"Personally modest and unassuming, George C. Schoenberger was one of those talented Louisianians whose work had escaped general public notice until last year when the greatest flood in the history of the Mississippi River centered attention on the men who bore the brunt of the battle against almost hopeless odds. In this crisis he emerged as an effective and skillful leader of the State's flood forces, whose command he had assumed two years before upon his appointment as Chief of State Engineers.

"Although disaster after disaster overwhelmed the flood-plain of the State, his courage never slackened, nor was there ever any flagging in his resolution to fight to the last to save every possible acre from destruction and ruin. His personal presence was felt wherever the water threatened in the far-flung battle line from the south bank of the Arkansas River to the Gulf of Mexico. In the long history of Mississippi River floods probably no more critical situation could be cited than that existing on the memorable night of Good Friday, 1927, when a rain and windstorm whipped the waters of the swollen Mississippi between New Orleans and Baton Rouge, shattering levees at a dozen different points, and threatening imminent disaster, which was averted only after several days of furious work. The holding of the levee lines on the Mississippi below Red River Landing until the Bayou des Glaisses crevasses had relieved the pressure in the main stream, was a notable achievement in itself.

"For the task which confronted him in the 1927 flood Mr. Schoenberger had been in training since his graduation from the State University in 1898, practically his entire time since then having been devoted to river work. He had espoused the spillway cause along with other notable engineers prior to the 1927 flood, and in the fight at Washington last fall and winter for adequate flood control legislation his views had marked effect. His unexpected death, following a year and a half of intense activity, first in the gruelling battle to hold the levees and afterwards at Washington, is a distinct loss to the State, to the service of which he had devoted the most useful years of his life."

He was married on January 8, 1902, to Podine Conrad Pope, of Baton Rouge, La. He is survived by his widow, three sons, George C., Jr., Ripley Pope, and Sidney Conrad, and one daughter, Podine.

Mr. Schoenberger was elected an Associate Member of the American Society of Civil Engineers on November 27, 1917.

EDWARD JOHN TULLY, Assoc. M. Am. Soc. C. E.*

DIED OCTOBER 27, 1928.

Edward John Tully was born at Omaha, Nebr., on June 23, 1893, the son of John Charles Tully and Gertrude (Stein) Tully. Most of his life was spent in Omaha, where he attended the public schools, finishing High School in 1909.

* Memoir prepared by Mrs. Genevieve Kennedy Tully, Omaha, Nebr.

He was employed by the Concrete Engineering Company, in Omaha, for several years, leaving to enter the Government service during the World War. During this period he was engaged in the Ship Building Department, first in Washington, D. C., and then in Philadelphia, Pa., later going to Camp Funston, Kansas. He was not sent overseas, however.

After returning to Omaha, Mr. Tully was employed by the Metropolitan Utilities District. He was Chief Draftsman for the design of the Omaha Filter Plant built in 1923 at a cost of \$600 000 for improving the water supply of the City. As Draftsman, he also assisted in the design of a 48-in. water main, 7 miles long, costing in excess of \$800 000. He designed the foundation and connections for two pumps with capacities of 30 000 000 gal. per day. After serving with the Metropolitan Utilities District for three and one-half years, under the supervision of C. D. Robinson, M. Am. Soc. C. E., Mr. Tully was forced to resign on account of illness.

In an unsuccessful attempt to regain his health, he spent the following two years, until his death, in California. When Mr. Tully left Omaha for Sierra Madre, Calif., he was engaged to be married to Genevieve Kennedy a sweetheart of several years. On account of his illness they thought it advisable to delay the marriage. On the advice of relatives, however, Miss Kennedy accompanied by Mr. Tully's aunt, went to California arriving on a Tuesday. The wedding ceremony was performed Wednesday afternoon and Mr. Tully passed away Saturday morning. Mr. Tully's remains were brought back to Omaha from California, and he was laid to rest in St. Mary's Cemetery, beside his mother and father.

Mr. Tully was very fond of outdoor sports. He was an expert swimmer and skater, and played baseball on several junior teams. He was very ambitious, possessing a keen, creative mind. During his school days he worked in his uncle's grocery store from five o'clock in the morning until it was time for him to go to school, and then again after school hours and on Saturdays. He also attended night school to further his engineering knowledge, and because he loved his work, was always studying during his sickness. He associated with older men rather than men of his own age, having such interest in his work that he felt they could help and advise him from experience. A man of likeable personality, he was quiet and reserved, but loved by all who knew him. Had he lived longer, he doubtless would have been very successful. Among the relationships that he cherished most, were his connections with the Society.

Mr. Tully was elected an Associate Member of the American Society of Civil Engineers on August 30, 1926.

CHARLES FREDERICK QUINCY, Affiliate, Am. Soc. C. E.*

DIED OCTOBER 1, 1927.

Charles Frederick Quincy, the son of George Henry and Mary Caroline (Sweetser) Quincy, was born in Newton, Mass., on July 16, 1856. On his

* Memoir compiled from information on file at the Headquarters of the Society.

father's side he was a direct descendant of Edmund Quincy who came from England in 1628 and settled in Massachusetts.

Mr. Quincy's elementary training was received at the Newton High School which, together with some time spent under the guidance of a private tutor, completed his education. It was during this period that he spent two years at sea, due to ill health, and was shipwrecked off the coast of Denmark.

In 1886, he entered the railroad supply business, a founder of the firm which first operated as the Dunham Manufacturing Company and, later, as the Q and C Company, of which he became President in 1887, and with which he was connected during the remainder of his active business life. Subsequent to 1922, Mr. Quincy gradually lessened his active participation in the work of the Company, and, in 1926, he was appointed Chairman of the Board of Directors, which office he held until his death. Mr. Quincy passed away on October 1, 1927, at his summer home, Centre Harbor, N. H.

He was also Chairman of the Board of the Dorr-Miller Differential Company, and the Miller Transmission Company and Vice-President and Director of the Kelvinator and Westchester Company.

Mr. Quincy was a member of the American Society of Mechanical Engineers, American Institute of Mining and Metallurgical Engineers, American Association for the Advancement of Science, and the American Society for Testing Materials.

He also held membership in the Railroad Club, the Greenwich Country Club, and the Lotos Club of New York, the Chicago Club, the New York Botanical Society, the New York Zoological Society, the National Geographical Society, and the Metropolitan Museum of Natural History.

Mr. Quincy's entry into the various mechanical societies was due to his success in presenting new railroad devices to the railroad trade. He was a great friend of the under-privileged boy, having been the first President of the Allendale Association at Lake Villa, Ill., and a member of the Board of the Boys Club Federation of New York and Hope Farm, at Verbank, N. Y.

On October 22, 1879, Mr. Quincy was married to Etta Molineaux Ives. He is survived by his widow and four children, Mrs. A. Q. Karcher, and Polly, Edmund, and Roger Bradshaw Quincy.

Mr. Quincy was elected an Affiliate of the American Society of Civil Engineers on February 4, 1896.

CHARLES FREDERICK QUINCY, ANNALS AND SOC. C. E. E.

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